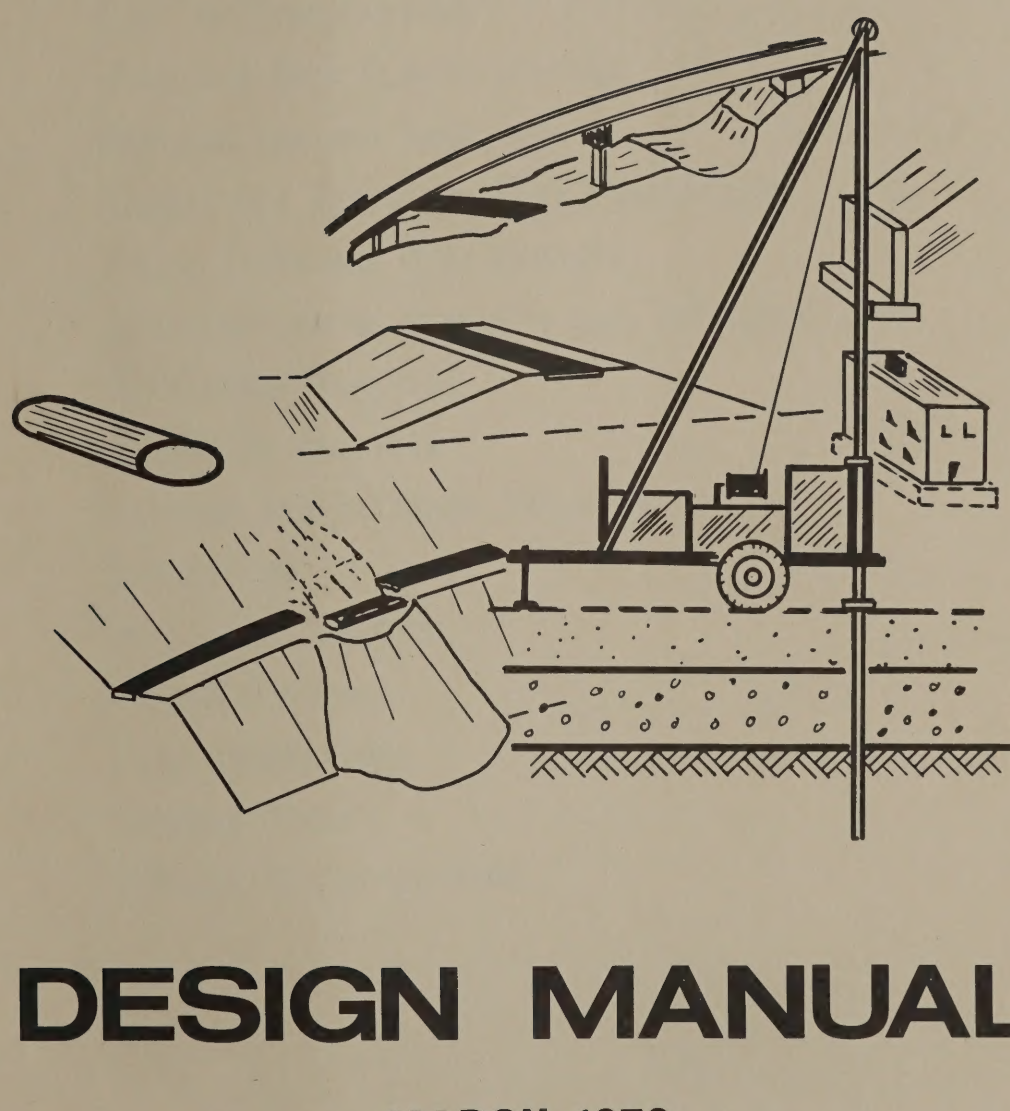
SOIL MECHANICS BUREAU



DESIGN MANUAL

MARCH 1978

SOIL MECHANICS DESIGN MANUAL

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(REVISED 3/78)

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SOIL AND ROCK CLASSIFICATION

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S.T.P. 2/74 AN ENGINEERING DESCRIPTION OF SOILS

M. G. CLINE & R. L. MARSHALL, SOILS OF NEW YORK LANDSCAPES

E. FERNAU, PHYSIOGRAPHIC INTERPRETATION OF SOILS

A

August 10, 1977

NEW YORK STATE DEPARTMENT OF TRANSPORATION
SOIL MECHANICS BUREAU

LIST OF PUBLICATIONS

7.41-1 SOIL TEST METHODS

STM-1	May, 1967	Test Method for Earthwork Compaction Control
STM-2	August, 1974	Test Method for the Grain-Size Analysis of Granular Soil Materials
STM-3	December, 1968	Test Method for Magnesium Sulfate Soundness of Granular Materials
STM-6	March, 1974	Test Method for Rapid Earthwork Compaction Control

7.41-2 SOIL DESIGN PROCEDURE

SDP-1	October, 1970	Computerized Analysis of the Stability of Earth Slopes
SDP-2	August, 1971	Bank and Channel Protective Lining Design Procedures
SDP-3	Spring, 1975 not issued	Design, Construction, and Maintenance of Recharge Basins

7.41-3 SOIL CONTROL PROCEDURES

SCP-1	April, 1969	Procedure for the Quality Control of Stockpiled Granular Materials
SCP-2	February, 1974	Procedure for the Control of Granular Materials
SCP-3	April, 1972	Settlement Gages and Settlement Stakes
SCP-4	June, 1974	Static Pile Load Test Manual
SCP-5	February, 1975	Inspection and Calibration of Soil Stabilization Plants

7.41-4 SPECIAL SOIL REPORTS

SR 69-1	December, 1969	Evaluation of Vibratory Compaction Equipment on Rockfills
SSR-1/72	October, 1972	Explanation and Guide to 1972 Survey Operations & Data - Pavement Serviceability Program, PIN E10513-701

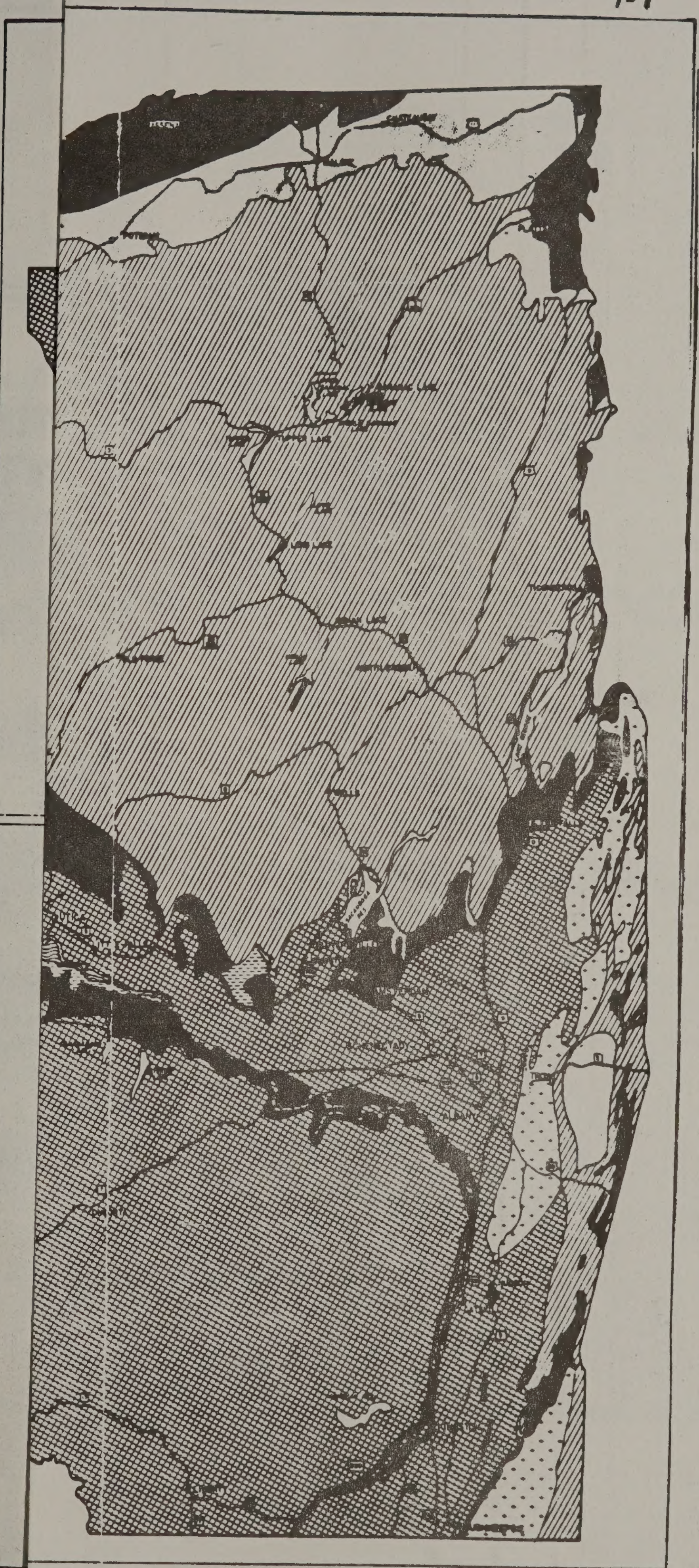
7.41-5 SOIL TEST PROCEDURE

STP-1	August, 1973	Test Procedures for Specific Surface Analysis
STP-2	May, 1975	Soils Description Procedure

7.41-6 SOILS ENGINEERING MANUALS

SEM-1	January, 1972	Bishop Slope Stability by Desk Top Computer
SEM-2	February, 1972	Navdocks Wedge Analysis by Desk Top Computer
SEM-3	February, 1972	Infinite Slope Analysis
SEM-4	May, 1972 obsolete	Procedure for Rating the Serviceability of Pavements
SEM-5 not issued	November, 1974	Computerized Modified Swedish Wedge Stability Analysis
SEM-6 not issued	April, 1974	Stability Curves for Embankments on Soft Soils
SEM-7	March, 1975	Distribution of Vertical Pressure Under Embankments

1-2
1-1



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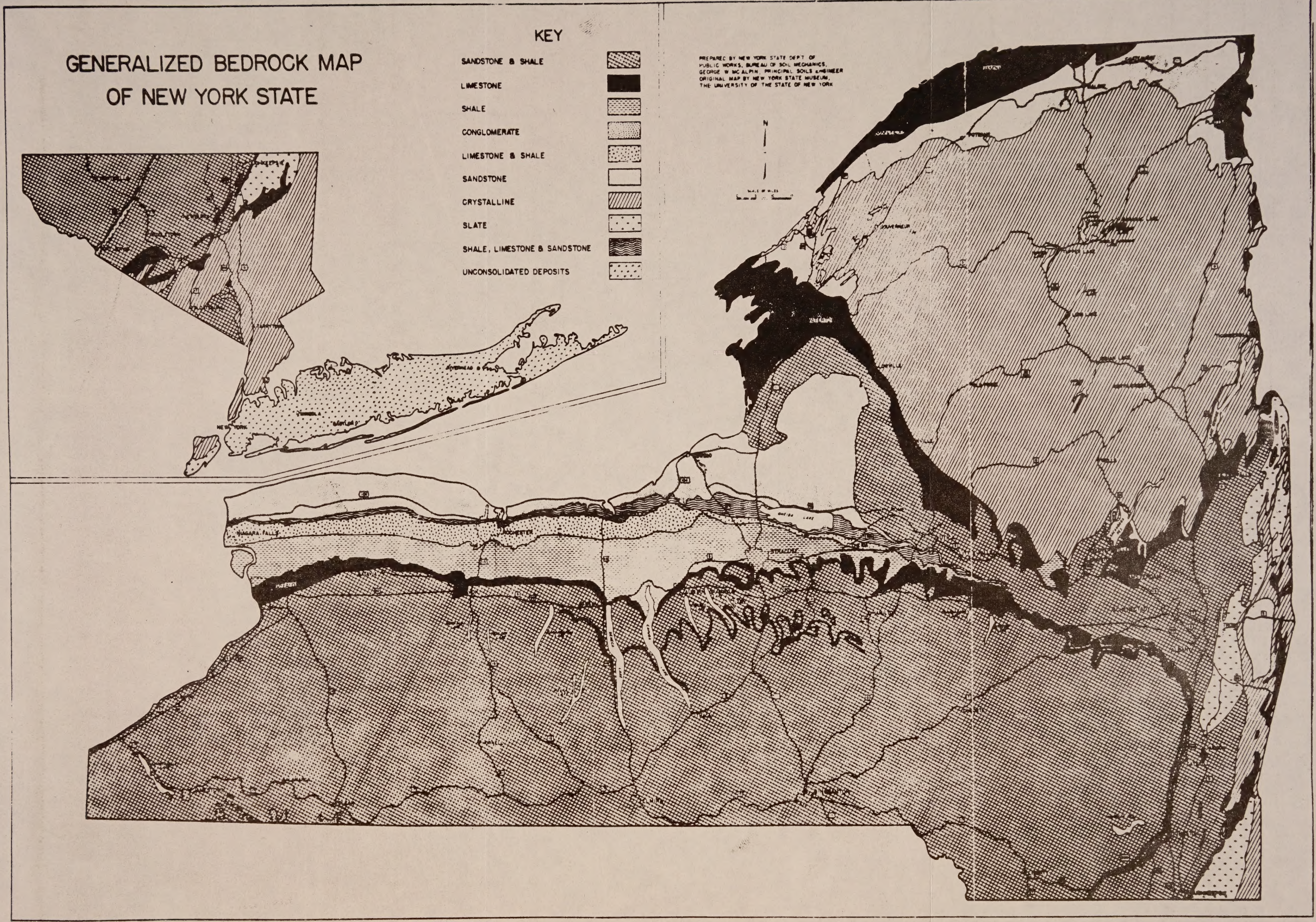
GENERALIZED BEDROCK MAP
OF NEW YORK STATE
KEY
SANDSTONE & SHALE
LIMESTONE
SHALE
CONGLOMERATE
LIMESTONE & SHALE
SANDSTONE
CRYSTALLINE
SLATE
SHALE, LIMESTONE & SANDSTONE
UNCONSOLIDATED DEPOSITS
PREPARED BY NEW YORK STATE DEPT OF
PUBLIC WORKS, BUREAU OF SOIL MECHANICS,
GEORGE W MC ALPIN, PRINCIPAL SOILS ENGINEER
ORIGINAL MAP BY NEW YORK STATE MUSEUM,
THE UNIVERSITY OF THE STATE OF NEW YORK
N
NIAGARA FALLS
ROCHESTER
SYRACUSE
NEW YORK

GLOSSARY

NDING – A LAYERING OF MINERALS OF DIFFERENT COLOR OR COMPOSITION. ESPECIALLY COMMON IN METAMORPHIC ROCKS.

LEAVAGE – SMOOTH PLANES CONSISTENTLY DEVELOPING IN MANY MINERALS. THE MINERALS SEPARATE ALONG THESE PLANES WHEN STRUCK A BLOW.

FFERVESCE – THE FIZZING OR BUBBLING ACTION WHEN A REACTIVE MINERAL IS PLACED IN CONTACT WITH MURIATIC OR HYDROCHLORIC ACID.

OLIATION – A STRUCTURAL TERM APPLIED TO METAMORPHIC ROCKS. WHEN THE FLAT MINERALS ARE IN PARALLEL ARRANGEMENT, THE ROCK IS FOLIATED.

RACTURE – SEPARATION OF MINERALS ALONG PLANES THAT ARE ROUGH AND NOT PARALLEL TO EACH OTHER. A RANDOM BREAKAGE OF MINERAL GRAINS.

OINTS – NATURAL AND USUALLY SMOOTH FRACTURES IN ROCK WHICH TEND TO SEPARATE THE ROCK INTO BLOCKS. JOINTS USUALLY BECOME LESS OBVIOUS WITH DEPTH.

AMINATED – A VERY THIN BEDDED ROCK. CLOSE-SPACED BEDDING PLANES AS IN A SHALE. LAMINATION PLANES ARE USUALLY PLANES OF WEAKNESS ALONG WHICH A ROCK WILL SEPARATE.

ETALLIC – THE LUSTER OF METAL.

ITREOUS – THE BRIGHT LUSTER OF QUARTZ OR GLASS.

RALS CHART

GE OR FRACTURE	MINERAL	DISTINGUISHING CHARACTERISTICS
AVAGE – UNEVEN RE.	QUARTZ	GLASSY APPEARANCE – HARDNESS.
T CLEAVAGE.	CALCITE	EFFERCESCES FREELY IN DILUTE HYDROCHLORIC ACID. FLAT CLEAVAGE FACES, HARDNESS.
GOOD CLEAVAGE – N FRACTURE.	FELDSPAR	COLOR – HARDNESS– ASSOCIATED WITH LIGHT COLORED ROCKS. BRIGHT CLEAVAGE FACES.
FRACTURE.	PYRITE (FOOL'S GOLD)	HARDNESS – COLOR.
EAVAGE – N FRACTURE.	SERPENTINE	COLOR – GREASY FEEL.
CT – PEELS LIKE ALONG CLEAVAGE S.	MICA BLACK – BIOTITE WHITE – MUSCOVITE	ELASTIC – WILL BEND AND SPRING BACK. PERFECT CLEAVAGE.
GOOD CLEAVAGE. N FRACTURE.	HORNBLENDE & PYROXENE	COLOR – CLEAVAGE FACES SHINE AS ROCK IS ROTATED IN LIGHT.
GOOD CLEAVAGE. N FRACTURE.	DOLOMITE	HARDNESS – WILL FEEBLY EFFERVESCE IN COOL DILUTE HYDROCHLORIC ACID.

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INTRODUCTION

A PROPER FIELD IDENTIFICATION OF ROCK SAMPLED BY THE CORE DRILL IS OF AID TO THE GEOLOGIST IN DETERMINING THE HISTORY, PROPERTIES, AND DURABILITY OF A ROCK AND AIDS THE ENGINEER IN EVALUATING THE SUITABILITY OF A ROCK AS APPLIED TO VARIOUS ENGINEERING WORKS.

GENERAL INTRODUCTION

ROCKS ARE GENERALLY DIVIDED INTO THREE GROUPS ON THE BASIS OF ORIGIN AND ARE THEN SUBDIVIDED ON THE BASIS OF COMPOSITION AND STRUCTURE.

IGNEOUS ROCKS - INCLUDE ALL ROCKS FORMED FROM MOLTEN MATERIAL. THESE ARE RECOGNIZED BY THE INTERLOCKING AND CRYSTALLINE APPEARANCE OF THE GRAINS AND THE LACK OF NOTICEABLE STRUCTURE IN THE ROCK SUCH AS BANDING AND FOLIATION.

SEDIMENTARY ROCKS - INCLUDE ALL ROCKS FORMED BY DEPOSITION ON THE CRUST OF THE EARTH BY AGENTS SUCH AS WATER AND WIND. THIS DEPOSITED MATERIAL USUALLY IS DERIVED FROM THE BREAKING DOWN OR WEATHERING OF OLDER ROCKS. THESE ARE MOST EASILY RECOGNIZED FROM THE ROUNDED APPEARANCE OF THE GRAINS, THE PRESENCE OF FOSSILS, OR THE LAYERED APPEARANCE OF THE ROCK AS A WHOLE.

METAMORPHIC ROCKS - INCLUDE BOTH IGNEOUS AND SEDIMENTARY ROCKS THAT HAVE BEEN SUBJECTED TO A CHANGE BY HEAT, PRESSURE, OR INVADING GASSES OR LIQUIDS. THESE ROCKS ARE MOST EASILY RECOGNIZED BY INTERLOCKING MINERAL GRAINS AND BY THEIR STRUCTURE, OR DEFINITE SYSTEMATIC AND ARRANGEMENT OF MINERAL GRAINS WHICH GIVES A LAYERED OR BANDED APPEARANCE TO THE ROCK. IGNEOUS AND METAMORPHIC ROCKS ARE SOMETIMES DIFFICULT TO DISTINGUISH.

ALL THREE OF THESE GENERAL ROCK TYPES ARE REPRESENTED IN NEW YORK STATE. AS CAN BE SEEN ON THE GEOLOGIC MAP, LARGE AREAS ARE COMPOSED PREDOMINATELY OF ONE ROCK TYPE, BUT IT MUST BE POINTED OUT THAT WITHIN THESE AREAS EITHER ONE OR BOTH OF THE OTHER TYPES MAY OCCUR AS BOULDERS IN THE OVERBURDEN OR AS LEDGE ROCK. THIS IS ESPECIALLY TRUE NEAR THE EDGES OF THE PREDOMINANT ROCK TYPE AREAS.

MINERALS

SINCE ANY ROCK IS AN AGGREGATE OF MINERALS, ITS FINAL CLASSIFICATION WILL INVOLVE THE IDENTIFICATION OF SOME OF THE MORE COMMON MINERALS. AS AN AID TO THIS IDENTIFICATION, A COMMON MINERAL CHART IS PROVIDED. ADDITIONAL ITEMS USEFUL IN IDENTIFYING MINERALS INCLUDE A POCKETKNIFE, A SAMPLE PIECE OF QUARTZ, A SMALL HAND LENS, AND A DROPPER TOP BOTTLE OF DILUTE (HALF ACID AND HALF WATER) HYDROCHLORIC (MURIATIC) ACID. THE USE OF THESE ITEMS IS DESCRIBED ON THE CHART.

FRACTURING AND WEATHERING

VERY FEW ROCK FORMATIONS OCCUR IN NATURE AS UNIFORM UNBROKEN MASSES. THE ROCKS ARE USUALLY FRACTURED, FAULTED OR JOINTED TO VARYING DEGREES. IN SOME PLACES THESE FRACTURES ARE CLOSE TOGETHER AND IN OTHERS THEY ARE FAR APART. THE SURFACES OF THE FRACTURES MAY BE ROUGH OR RELATIVELY SMOOTH. THEY MAY BE COATED WITH MINERAL MATERIAL. SOME OF THE FRACTURES IN A ROCK CORE MAY BE NOTED TO HAVE A DEFINITE AND SYSTEMATIC RELATIONSHIP TO THE ANGLE AT WHICH THE CORE IS BEING DRILLED. ANY PREDOMINATE AND PERSISTENT SET OF FRACTURE SURFACES SHOULD BE NOTED ON THE LOG. ALSO, ANY LOSS OF WATER SHOULD BE NOTED, AS THIS IS AN INDICATION OF WHETHER OR NOT THE ENCOUNTERED FRACTURES ARE OPEN OR TIGHT.

MOST ROCK IS WEATHERED TO VARYING DEGREES NEAR THE SURFACE AND BECOMES FRESHER AS IT IS PENETRATED. WEATHERING PENETRATES ROCK MOST RAPIDLY ALONG FRACTURES AND FRACTURE ZONES. IN SOME AREAS OF THE STATE WEATHERING HAS PENETRATED SEVERAL HUNDRED FEET ALONG SUCH ZONES, WHEREAS, IN ADJACENT SIMILAR BUT UNFRACTURED ROCK IT HAS PENETRATED ONLY A FEW INCHES. THE DEGREE OF WEATHERING SHOULD BE NOTED ON THE DRILL LOG. BADLY WEATHERED ROCK IS THAT WHICH CAN BE CRUMBLED BY HAND OR BY ONLY LIGHT PRESSURE WITH A HAMMER. PARTIALLY WEATHERED ROCK IS THAT IN WHICH ONLY ONE MINERAL OR SET OF MINERALS IS BADLY WEATHERED. SLIGHTLY WEATHERED ROCK IS BLEACHED, STAINED, OR DISCOLORED, WITH THE STAINING PENETRATING THE ROCK FOR ONLY A SHORT DISTANCE. UNWEATHERED ROCK GIVES A FRESH APPEARANCE IN THE CORE. IN SOME CASES, ONLY SURFACE STAINING ALONG FRACTURE SURFACES WILL BE NOTED AND THE REMAINDER OF THE ROCK WILL APPEAR FRESH AND UNWEATHERED.

RECOVERY

THE IMPORTANCE OF RECOVERY OF CORE CANNOT BE OVER-EMPHASIZED. OFTEN IT IS THE CORE THAT IS LOST THAT IS MOST CRITICAL TO THE DESIGN OF A CUT OR STRUCTURE. ALL SOFT SEAMS AND DECAYED LEDGE SHOULD BE SAMPLED IN AN EFFORT TO OBTAIN RECOVERY APPROACHING 100 PERCENT. IF DRILL RODS DROP AND NO SAMPLE CAN BE OBTAINED IN THE APPARENT VOID, THIS FACT SHOULD BE NOTED ON THE LOG. ANY ADDITIONAL REASONS FOR POOR RECOVERY AND DEPTH AT WHICH THE POOR RECOVERY OCCURS SHOULD ALSO BE RECORDED.

CHECK LIST FOR INFORMATION ON DRILL LOGS

THE FOLLOWING ITEMS SHOULD BE NOTED ON DRILL LOGS WHERE ROCK IS ENCOUNTERED:

1. ROCK TYPES AND DEPTH OF CHANGES.
2. DEGREE AND DEPTH OF WEATHERING.
3. DEGREE AND DEPTH OF FRACTURING.
4. NUMBER OF PIECES PER RUN AND LIMITS OF SIZES.
5. PERCENT OF RECOVERY.
6. PROBABLE REASONS FOR POOR RECOVERY.
7. COLOR OF WATER RETURN AND DEPTH OF CHANGES.
8. DEPTH OF WATER LOSSES.

GLOSSARY

BANDING - A LAYERING OF MINERALS OF DIFFERENT COLOR OR COMPOSITION. ESPECIALLY COMMON IN METAMORPHIC ROCKS.

CLEAVAGE - SMOOTH PLANES CONSISTENTLY DEVELOPING IN MANY MINERALS. THE MINERALS SEPARATE ALONG THESE PLANES WHEN STRUCK A BLOW.

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JOINTS - NATURAL AND USUALLY SMOOTH FRACTURES IN ROCK WHICH TEND TO SEPARATE THE ROCK INTO BLOCKS. JOINTS USUALLY BECOME LESS OBVIOUS WITH DEPTH.

LAMINATED - A VERY THIN BEDDED ROCK. CLOSE-SPACED BEDDING PLANES AS IN A SHALE. LAMINATION PLANES ARE USUALLY PLANES OF WEAKNESS ALONG WHICH A ROCK WILL SEPARATE.

METALLIC - THE LUSTER OF METAL.

VITREOUS - THE BRIGHT LUSTER OF QUARTZ OR GLASS.

COMMON MINERALS CHART

COLOR	HARDNESS	LUSTER	CLEAVAGE OR FRACTURE	MINERAL	DISTINGUISHING CHARACTERISTICS
WHITE, LIGHT BLUE, PINK, TRANSPARENT.	WILL SCRATCH A KNIFE BLADE AND GLASS.	VITREOUS	NO CLEAVAGE - UNEVEN FRACTURE.	QUARTZ	GLASSY APPEARANCE - HARDNESS.
WHITE, BLUE, PINK, TRANSPARENT.	EASILY SCRATCHED WITH A KNIFE.	VITEROUS	PERFECT CLEAVAGE.	CALCITE	EFFERCESCES FREELY IN DILUTE HYDROCHLORIC ACID. FLAT CLEAVAGE FACES, HARDNESS.
WHITE, GREY, PINK, GREEN, RED.	CANNOT BE SCRATCHED WITH A KNIFE; CAN BE SCRATCHED WITH QUARTZ.	VITREOUS	VERY GOOD CLEAVAGE - UNEVEN FRACTURE.	FELDSPAR	COLOR - HARDNESS- ASSOCIATED WITH LIGHT COLORED ROCKS. BRIGHT CLEAVAGE FACES.
BRASS YELLOW.	HARD - WILL SCRATCH A KNIFE.	METALLIC	UNEVEN FRACTURE.	PYRITE (FOOL'S GOLD)	HARDNESS - COLOR.
GREEN, YELLOW-GREEN.	SOFT - EASILY SCRATCHED WITH A KNIFE.	WAXY - GREASY	NO CLEAVAGE - UNEVEN FRACTURE.	SERPENTINE	COLOR - GREASY FEEL.
WHITE, LIGHT BROWN, BLACK.	CAN BE SCRATCHED WITH A PENNY.	VITREOUS	PERFECT - PEELS LIKE PAPER ALONG CLEAVAGE PLANES.	MICA BLACK - BIOTITE WHITE - MUSCOVITE	ELASTIC - WILL BEND AND SPRING BACK. PERFECT CLEAVAGE.
GREENISH BLACK, BLACK.	SCRATCHED WITH A KNIFE WITH DIFFICULTY. EASILY SCRATCHED WITH QUARTZ.	VITREOUS	VERY GOOD CLEAVAGE. UNEVEN FRACTURE.	HORNBLENDE & PYROXENE	COLOR - CLEAVAGE FACES SHINE AS ROCK IS ROTATED IN LIGHT.
WHITE, GREY.	SCRATCHED WITH A KNIFE.	VITREOUS	VERY GOOD CLEAVAGE. UNEVEN FRACTURE.	DOLOMITE	HARDNESS - WILL FEEBLY EFFERVESCE IN COOL DILUTE HYDROCHLORIC ACID.

RTZ AND FELDSPAR PRE- PINK, OR GREEN FELD-
A SALT AND PEPPER OXENE. MINERAL THE ROCK. DRILLS SPARS.
DISTRIBUTED. EASIER ARANCE. WHITE OR
RTZ AND IF SO IS . WHITE OR GREY
GRANITE. WHITE
OCK. EASIER TO

ROKEN. A HARD IFE BLADE OR
TINCTLY CRYSTALLINE. PHITE FLAKES. EASY
WITH A PENKNIFE.
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OCHLORIC ACID. OFTEN TE. DRILLS EASILY. SHINE AS ROCK IS ICULT MINERAL TO
DILUTE HYDROCHLORIC E IN ANY PROPORTION.
Y LAMINATED. SOMETIMES WILL NOT EFFERVESCE IN OL'S GOLD) AND SOMETIMES CAN BE SCRATCHED WITH A

IGNEOUS
(SEE MAP AND TEXT)

COLOR AND GROUP	MINERALS	TEXTURE	ROCK	DIAGNOSTIC PROPERTIES
LIGHT COLORED GROUP. THIS GROUP CAN BE SUB-DIVIDED IN THE FIELD ON THE BASIS OF GRAIN-SIZE AND QUARTZ CONTENT.	QUARTZ, FELDSPAR, MICA	VERY COARSE GRAINED	PEGMATITE	LARGE GRAIN-SIZE. CAN CONTAIN MANY MINERALS BUT QUARTZ AND FELDSPAR PREDOMINATE. DRILLS WITH MODERATE DIFFICULTY. WHITE, PINK, OR GREEN FELDSPAR.
	QUARTZ FELDSPAR MICA HORNBLENDE PYROXENE	COARSE TO FINE GRAINED, INTERLOCKING GRAINS	GRANITE	LIGHT COLOR, WHITE, GREY, OR PINK. SOMETIMES GIVES A SALT AND PEPPER APPEARANCE DUE TO MINOR AMOUNTS OF AMPHIBOLE AND PYROXENE. MINERAL GRAINS EQUALLY DISTRIBUTED. QUARTZ A MAJOR PART OF THE ROCK. DRILLS WITH MODERATE DIFFICULTY. WHITE, PINK OR GREEN FELDSPARS.
	FELDSPAR MICA HORNBLENDE PYROXENE	COARSE TO FINE GRAINED, INTERLOCKING GRAINS	SYENITE	LIGHT COLOR, LITTLE OR NO QUARTZ. MINERALS EQUALLY DISTRIBUTED. EASIER TO DRILL THAN GRANITE. OFTEN A SALT AND PEPPER APPEARANCE. WHITE OR PINK FELDSPARS.
DARK COLORED BASIC IGNEOUS ROCK GROUP. THIS GROUP CAN BE SUB-DIVIDED IN THE FIELD ONLY WITH DIFFICULTY.	FELDSPAR BLACK MICA HORNBLENDE PYROXENE	COARSE TO FINE GRAINED, INTERLOCKING GRAINS	DIORITE	OFTEN HAS A GREENISH APPEARANCE. OFTEN CONTAINS QUARTZ AND IF SO IS CALLED QUARTZ DIORITE. EASIER TO DRILL THAN GRANITE. WHITE OR GREY FELDSPARS.
	FELDSPAR HORNBLENDE PYROXENE	COARSE INTERLOCKING GRAINS	GABBRO	COARSE GRAIN SIZE, DARK COLOR. EASIER TO DRILL THAN GRANITE. WHITE OR GRAY FELDSPARS.
	FELDSPAR HORNBLENDE PYROXENE	MEDIUM TO FINE INTERLOCKING GRAINS	DIABASE	FINE GRAIN-SIZE, DARK COLOR. COMMONLY CALLED TRAP ROCK. EASIER TO DRILL THAN GRANITE.

METAMORPHIC
(SEE MAP AND TEXT)

STRUCTURE	COLOR	COMPOSITION	ROCK	ORIGINAL ROCK	DIAGNOSTIC PROPERTIES
MASSIVE	WHITE GREY RED	QUARTZ	QUARTZITE	SANDSTONE	GLASSY APPEARANCE. FRACTURES ACROSS THE GRAINS WHEN BROKEN. A HARD DENSE ROCK AND DIFFICULT TO DRILL. WILL SCRATCH A KNIFE BLADE OR GLASS.
	WHITE GREY	CALCITE DOLOMITE	MARBLE	LIMESTONE	EFFERVESCES IN DILUTE HYDROCHLORIC ACID. USUALLY DISTINCTLY CRYSTALLINE. CAN BE SCRATCHED WITH A PENKNIFE. OFTEN CONTAINS GRAPHITE FLAKES. EASY TO DRILL.
	GREEN	SERPENTINE	SERPENTINE	PERIDOTITE	YELLOWISH GREEN COLOR. SOFT. CAN BE EASILY SCRATCHED WITH A PENKNIFE. USUALLY IS FRACTURED. EASY TO DRILL.
BANDED	GREY GREEN PINK	QUARTZ FELDSPAR MICA	GNEISS	GRANITE	BANDED APPEARANCE. LONG DIMENSIONS OF MINERALS ALIGNED. MODERATELY DIFFICULT TO DRILL.
FOLIATED (COARSE GRAINED)	GREEN GREY BROWN	MICA QUARTZ FELDSPAR	SCHIST	SLATE	FLAT MINERALS ALIGNED AND EASILY SEEN WITH NAKED EYE. COMPOSED GENERALLY OF A LARGE PERCENTAGE OF MICAS.
FOLIATED (FINE GRAINED)	GREY RED GREEN PURPLE	MICA QUARTZ FELDSPAR	SLATE	SHALE	FLAT MINERALS ALIGNED AND SEEN WITH THE NAKED EYE WITH DIFFICULTY. CONFUSED WITH SHALE. SHALE HAS A DULL SURFACE, WHEREAS THE SURFACE OF SLATE IS SHINY. DRILLS EASILY.

SEDIMENTARY
(SEE MAP AND TEXT)

STRUCTURE	COLOR	PARTICLE SHAPE AND SIZE	COMPOSITION	CEMENT	ROCK	DIAGNOSTIC PROPERTIES
MASSIVE OR LAYERED	RED GREY BROWN	VERY COARSE TO FINE ROUNDED GRAINS	ROCK FRAGMENTS	IRON CALCITE SILICA CLAY	CONGLO-MERATE	COARSE GRAIN-SIZE. ROUNDED PEBBLES UP TO AN INCH OR MORE IN DIAMETER COMPOSING THE MAJOR PART OF THE ROCK. MODERATE TO DIFFICULT TO DRILL, DEPENDING ON TYPE AND STRENGTH OF CEMENT.
	RED GREY BROWN	MEDIUM TO FINE ROUNDED GRAINS	ROCK FRAGMENTS QUARTZ FELDSPAR	IRON CALCITE SILICA CLAY	SAND-STONE	SAND-SIZE ROUNDED PARTICLES. MODERATE TO DIFFICULT TO DRILL, DEPENDING ON TYPE AND STRENGTH OF CEMENT. USUALLY BREAKS AROUND THE GRAINS.
	LIGHT GREY TO BLACK	ANGULAR TO ROUNDED GRAINS LARGELY INTER-LOCKING	CALCITE	CALCITE SILICA CLAY	LIME-STONE	DENSE STONE WHICH EFFERVESCES FREELY IN DILUTE HYDROCHLORIC ACID. OFTEN CONTAINS FOSSILS. OFTEN IS VEINED WITH WHITE CALCITE. DRILLS EASILY. CAN BE SCRATCHED WITH A KNIFE. FLAT CALCITE FACES SHINE AS ROCK IS ROTATED IN LIGHT. SOMETIMES CONTAINS CHERT, A DIFFICULT MINERAL TO DRILL.
	WHITE TO DARK GREY	ANGULAR TO ROUNDED GRAINS LARGELY INTER-LOCKING	DOLOMITE CALCITE	CALCITE SILICA CLAY	DOLOMITE	SIMILAR TO LIMESTONE BUT EFFERVESCES ONLY FEEBLY IN DILUTE HYDROCHLORIC ACID. LIMESTONE AND DOLOMITE MAY BE FOUND IN NATURE IN ANY PROPORTION. DRILLS EASILY. CAN BE SCRATCHED WITH A KNIFE.
LAMINATED	BLACK RED GREY GREEN PURPLE	VERY FINE ANGULAR PARTICLES	CLAY QUARTZ FELDSPAR	CLAY CALCITE SILICA	SHALE	PRESENTS A DULL APPEARANCE AND IS USUALLY DISTINCTLY LAMINATED. SOMETIMES CONFUSED WITH FINE GRAINED LIMESTONE, BUT WHEN PURE WILL NOT EFFERVESCE IN ACID, OFTEN CONTAINS SCATTERED GRAINS OF PYRITE (FOOL'S GOLD) AND SOMETIMES IS VEINED WITH CALCITE OR QUARTZ. DRILLS EASILY. CAN BE SCRATCHED WITH A KNIFE.

SECTION 2

EXPLORATION AND SAMPLING

<u>PAGES</u>

<u>MANUALS</u>

DRILL ROD

NOMENCLATURE	DIA.	LENGTH
F	1.1	2'-3'-5'
AW	1.7	1'-2'-5'-10'
NW	2.6	1'-2'-5'-10'

CASING W SERIES (FLUSH JOINT)

NOMENCLATURE	I.D.	O.D.	LENGTH
BW	2.4	2.9	1'-2'-5'
HW	4.0	4.5	1'-2'-5'

CASING X SERIES (FLUSH COUPLING)

NOMENCLATURE	I.D.	O.D.	COUPLING I.D.	LENGTH
BX	2.6	2.9	2.4	2'-5'
NX	3.2	3.5	3.0	2'-5'

DRIVE PIPE

NOMENCLATURE	I.D.	O.D.	COUPLING O.D.	LENGTH
2½" Extra Heavy	.19'	.23'	3½" Random	Random
4" Extra Heavy	.31'	.36'	.43'	5'

CORE BARRELS & BITS

NOMENCLATURE	HOLE SIZE	CORE SIZE
AWG	1.9	1.2
BWG	2.4	1.7
NWG	3.0	2.2
NXD3	3.0	2.1
HWG	3.9	3.0

SAMPLE SPOONS

NOMENCLATURE	O.D.	I.D.	HEAD CONN.
2"	2.0	1.5	AW
3"	3.0	2.5	NW
3½"	3.5	3.0	NW

GENERAL PURPOSE:

Shelby tube samples are ta
laboratory testing to obtain t
of fine grained soils containi
organic material. It is extre
pressed and transported with a
sampling and careless handling
results that could result in u

1. MEASUREMENTS

   A. All measurements shoul
   This includes measurem
   and sampler and length

2. CLEANING OUT HOLE

   A. The hole should be cle

   B. Clean out should be do
   If material cannot be
   conventional type tool

   C. Do not use bottom disc
   Frequently samples are
   center.

3. RATE OF PRESS

   A. Shelby tube should be
   operation.

   B. Make sure ports are cl

   C. For soft soils wait 5
   one complete turn to s

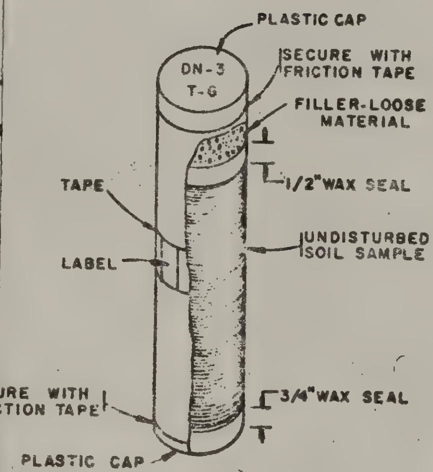
4. AMOUNT OF SAMPLE PRESSED

   A. Overpressing causes sa

   B. For stationary piston
   1.5 feet. After lower
   positive locking devic
   should always be used
   removal of sample from
   actuating rod to break
   the sampler.

   C. There may be conditio
   obtained. If recovery
   sample in tube. If re
   sample from shelby tub
   not mash or remold sa
   Bureau with shelby

   D. If sample cannot be p



SHELBY TUBE

NOT TO SCALE

DEPA

INSTRUCTIONS FOR UNDISTURBED SAMPLING OPERATIONS

DRILL ROD

NOMENCLATURE	DIA.	LENGTH
F	1.1	2'-3'-5'
AW	1.7	1'-2'-5'-10'
NW	2.6	1'-2'-5'-10'

CASING W SERIES (FLUSH JOINT)

NOMENCLATURE	I.D.	O.D.	LENGTH
BW	2.4	2.9	1'-2'-5'
HW	4.0	4.5	1'-2'-5'

CASING X SERIES (FLUSH COUPLING)

NOMENCLATURE	I.D.	O.D.	COUPLING I.D.	LENGTH
BX	2.6	2.9	2.4	2'-5'
NX	3.2	3.5	3.0	2'-5'

DRIVE PIPE

NOMENCLATURE	I.D.	O.D.	COUPLING O.D.	LENGTH
2½" Extra Heavy	.19'	.23'	3½" Random	Random
4" Extra Heavy	.31'	.36'	.43'	5'

CORE BARRELS & BITS

NOMENCLATURE	HOLE SIZE	CORE SIZE
AWG	1.9	1.2
BWG	2.4	1.7
NWG	3.0	2.2
NXD3	3.0	2.1
HWG	3.9	3.0

SAMPLE SPOONS

NOMENCLATURE	O.D.	I.D.	HEAD CONN.
2"	2.0	1.5	AW
3"	3.0	2.5	NW
3½"	3.5	3.0	NW

GENERAL PURPOSE:

Shelby tube samples are taken to obtain undisturbed samples for laboratory testing to obtain the strength and settlement properties of fine grained soils containing silt and clay and in some cases organic material. It is extremely important that the samples be pressed and transported with a minimum amount of disturbance. Poor sampling and careless handling of samples causes misleading test results that could result in uneconomical designs.

1. MEASUREMENTS

   A. All measurements should be made and recorded accurately. This includes measurements of drive pipe, drilling tools and sampler and length and recovery of sample pressed.

2. CLEANING OUT HOLE

   A. The hole should be cleaned out thoroughly before sampling.

   B. Clean out should be done with a clean-out jet type auger. If material cannot be removed with these tools the use of conventional type tools will be permitted.

   C. Do not use bottom discharge chopping bit in soft soils. Frequently samples are received with jet holes in the center.

3. RATE OF PRESS

   A. Shelby tube should be pressed by a smooth continuous operation.

   B. Make sure ports are clean.

   C. For soft soils wait 5 to 15 minutes before rotating sampler one complete turn to shear end of sample.

4. AMOUNT OF SAMPLE PRESSED

   A. Overpressing causes sample disturbance. Do not overpress.

   B. For stationary piston type sampler maximum press should be 1.5 feet. After lowering sampler to bottom of hole a positive locking device to hold actuating rod stationary should always be used before pressing sample. After removal of sample from drill hole be sure to back off actuating rod to break vacuum before removing piston from the sampler.

   C. There may be conditions where full recovery may not be obtained. If recovery is greater than .5 feet retain sample in tube. If recovery is less than .5 feet remove sample from shelby tube and cut sample to fit in jar. Do not mash or remold sample. Forward jar to the Soil Mechanics Bureau with shelby tube samples.

   D. If sample cannot be pressed use brass liner sampler.

5. WAX PLUGS

   A. A wax plug approximately 3/4 inches in thickness should be poured in the bottom of tube (cutting edge end). The laboratory uses this plug as a piston in pushing the sample out.

   B. A wax seal not greater than 1/2 inch in thickness should be poured in the top. Please avoid thick plugs as they are difficult to remove.

   C. The wax should consist of approximately equal parts of micro-crystalline wax (yellow or brown) and paraffin (white). Never use micro-crystalline wax alone on tubes or jars as it is very difficult to remove.

6. SEALING TUBE

   A. Any space remaining between the wax plug and the top of the tube should be filled with light loose material such as sawdust.

   B. Plastic caps should be placed on each end of tube and secured to tube with friction tape. No waxing is required on plastic caps.

7. LABELING TUBE

   A. Print on top cap with magic marker the hole number and sample number. This is precaution against label being lost.

   B. Paste and tape label on tube approximately ten inches from top of tube with friction tape or masking tape. The label should include project, sample number, project name, hole number, station, offset and depth of sample.

8. TRANSPORTING SHELBY TUBES

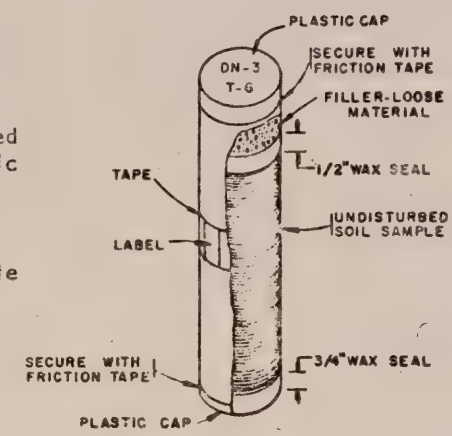
   A. Careful sampling in field can be ruined by careless handling of samples. Transport and ship sample in upright position. Carrying racks with proper cushioning should be used. Design for rack is available from Bureau of Soil Mechanics. Handle tubes carefully and avoid dropping, rolling and banging together.

   B. Protect sample from freezing or excessive heat.

SUBSURFACE EXPLORATION AND SOIL SAMPLE NUMBERING SYSTEM

1. Drill hole numbers or combinations of letters and numbers should not be repeated on any one project. Standard abbreviations are DA - drill hole, DN - undisturbed sample drill hole, AH - auger hole, and RP - retractable sampler hole.

2. All samples from any given hole should be numbered consecutively from the top to the bottom of hole. Standard abbreviations for samples are J - jar, T - shelby tube, BL - brass liner, and B - bag, do not change consecutive numbers if type of samples obtained change in hole.



SHELBY TUBE

NOT TO SCALE

STATE OF NEW YORK DEPARTMENT OF TRANSPORTATION SOIL MECHANICS BUREAU	PREPARED BY: ____ CHECKED BY: ____	DRAWING NO. SM 1627R2	REVISIONS: R1-CHANGES AND ADDITIONS 5/12/65 R2-CHANGES AND ADDITIONS 12/14/77	INSTRUCTIONS FOR UNDISTURBED SAMPLING OPERATIONS

SECTION 3

LABORATORY TESTS AND TEST PROPERTIES

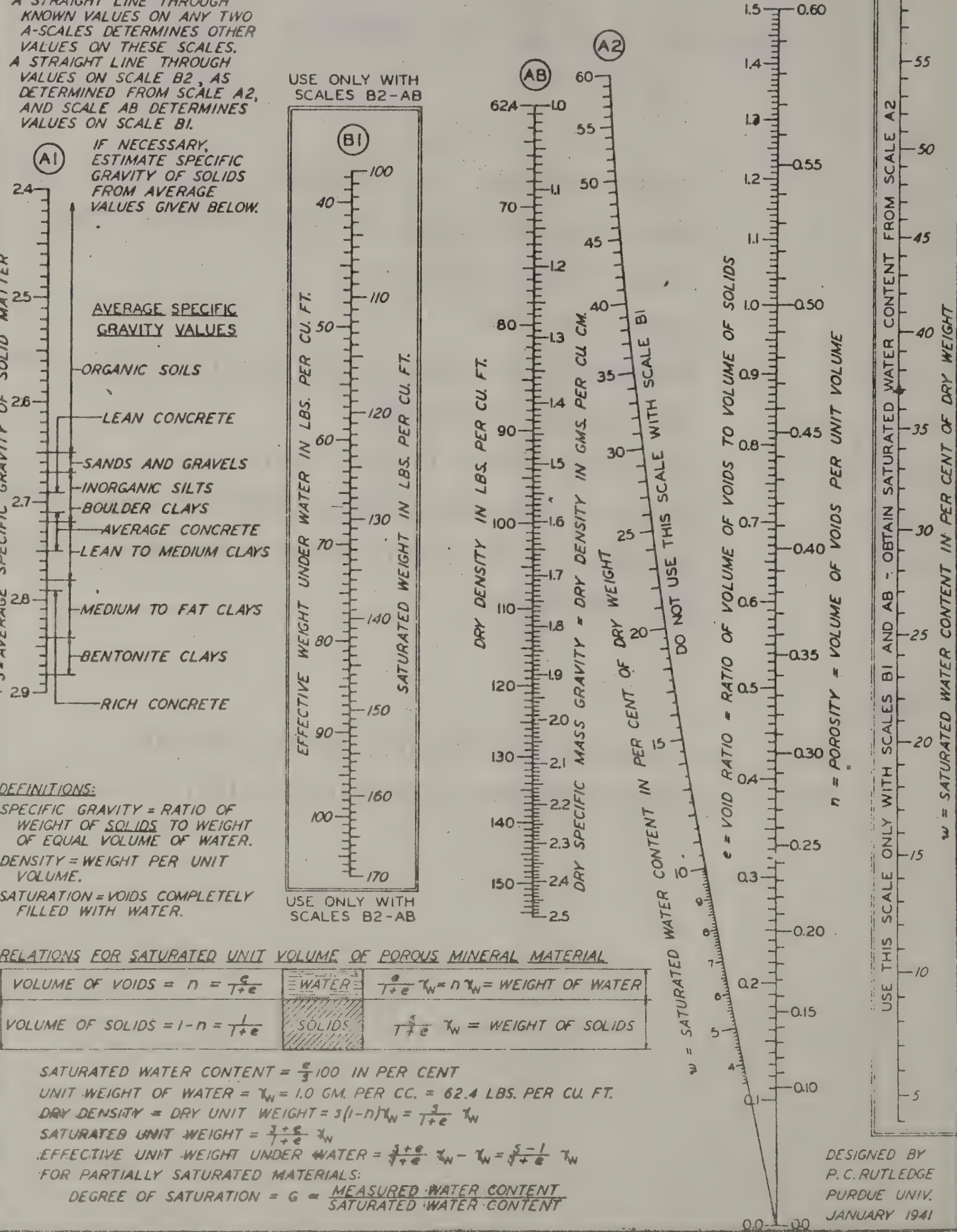
NOMOGRAPHIC CHART FOR FUNDAMENTAL VOLUMETRIC AND GRAVIMETRIC RELATIONS FOR SOILS, CONCRETES, AND POROUS MINERAL MATERIALS

DIRECTIONS FOR USE:

A STRAIGHT LINE THROUGH KNOWN VALUES ON ANY TWO A-SCALES DETERMINES OTHER VALUES ON THESE SCALES.

A STRAIGHT LINE THROUGH VALUES ON SCALE B2, AS DETERMINED FROM SCALE A2, AND SCALE AB DETERMINES VALUES ON SCALE B1.

IF NECESSARY, ESTIMATE SPECIFIC GRAVITY OF SOLIDS FROM AVERAGE VALUES GIVEN BELOW.



DEFINITIONS:

SPECIFIC GRAVITY = RATIO OF WEIGHT OF SOLIDS TO WEIGHT OF EQUAL VOLUME OF WATER.

DENSITY = WEIGHT PER UNIT VOLUME.

SATURATION = VOIDS COMPLETELY FILLED WITH WATER.

RELATIONS FOR SATURATED UNIT VOLUME OF POROUS MINERAL MATERIAL

VOLUME OF VOIDS = $n = \frac{e}{1+e}$	WATER	$\frac{e}{1+e}\gamma_W = n\gamma_W$ = WEIGHT OF WATER
VOLUME OF SOLIDS = $1-n = \frac{1}{1+e}$	SOLIDS	$\frac{s}{1+e}\gamma_W$ = WEIGHT OF SOLIDS

SATURATED WATER CONTENT = $\frac{e}{s}100$ IN PER CENT

UNIT WEIGHT OF WATER = γ_W = 1.0 GM. PER CC. = 62.4 LBS. PER CU. FT.

DRY DENSITY = DRY UNIT WEIGHT = $s(1-n)\gamma_W = \frac{s}{1+e}\gamma_W$

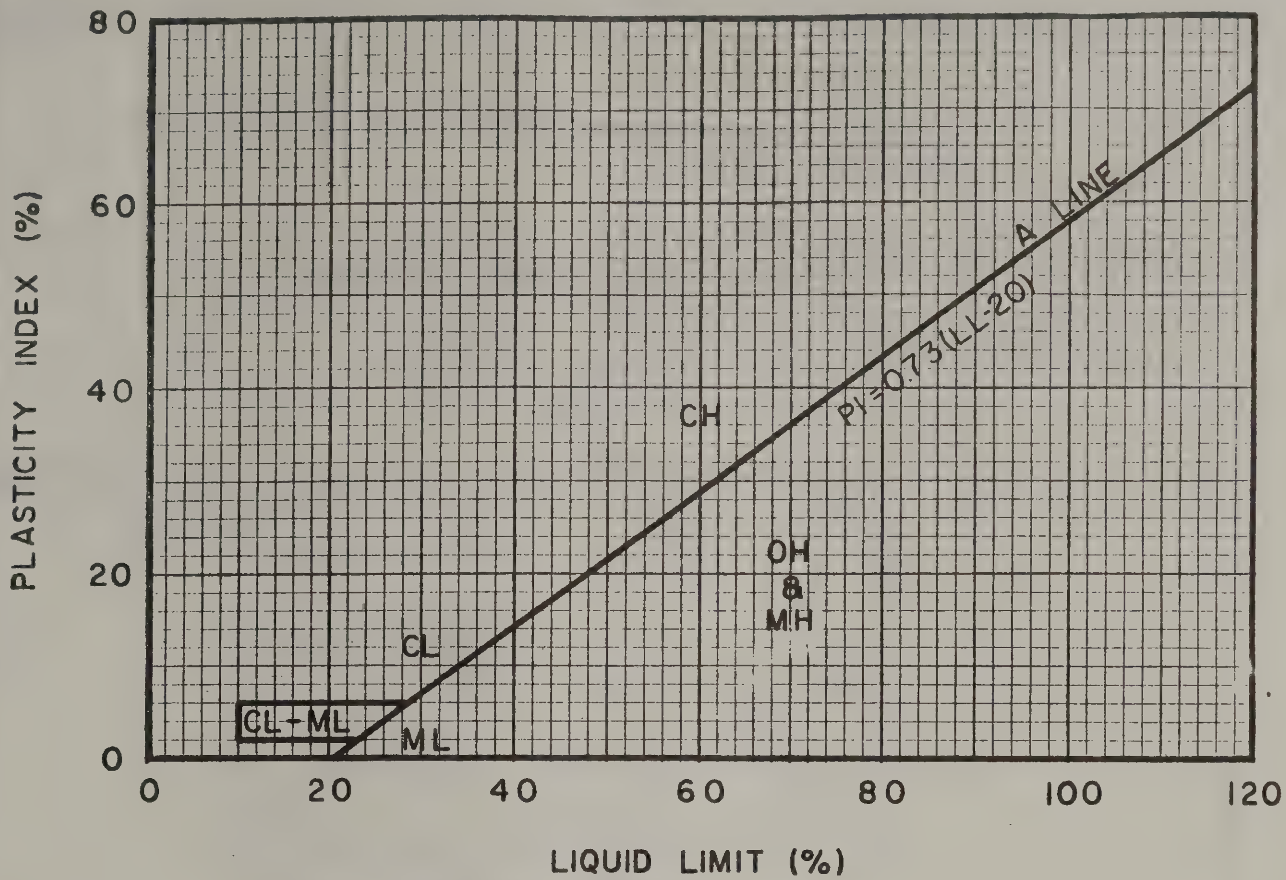
SATURATED UNIT WEIGHT = $\frac{s+e}{1+e}\gamma_W$

EFFECTIVE UNIT WEIGHT UNDER WATER = $\frac{s+e}{1+e}\gamma_W - \gamma_W = \frac{s-1}{1+e}\gamma_W$

FOR PARTIALLY SATURATED MATERIALS:

DEGREE OF SATURATION = $G = \frac{\text{MEASURED WATER CONTENT}}{\text{SATURATED WATER CONTENT}}$

DESIGNED BY
P. C. RUTLEDGE
PURDUE UNIV.
JANUARY 1941



SYMBOLS

CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.

ML Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.

CH Inorganic clays of high plasticity, fat clays.

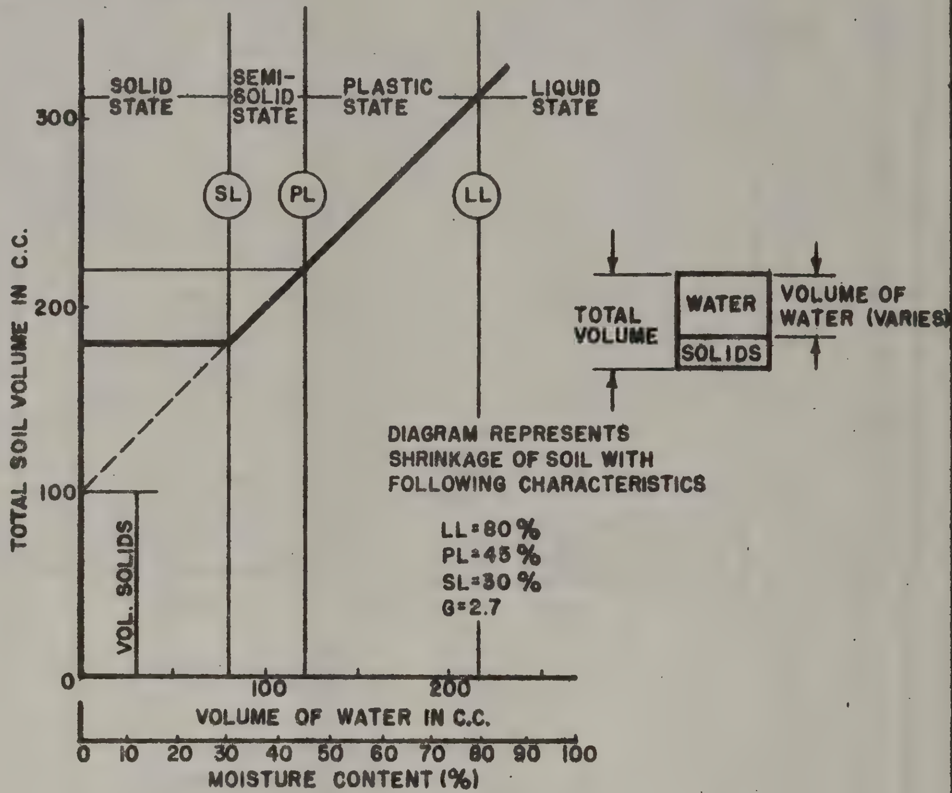
OH Organic clays of medium to high plasticity, organic silts.

MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.

When comparing soils at equal liquid limits, the toughness and dry strength increases and the permeability and rate of volume change decreases as the plasticity index increases.

PLASTICITY CHART

DIAGRAM SHOWING SHRINKAGE OF CLAY SOIL WITH DECREASE IN MOISTURE CONTENT.



SOIL STATE	ATTERBERG LIMITS	INDEXES
LIQUID		
	LIQUID LIMIT, LL	
PLASTIC		PLASTIC INDEX PI = LL - PL
	PLASTIC LIMIT, PL	
SEMISOLID		SHRINKAGE INDEX SI= PL-SL
	SHRINKAGE LIMIT, SL	
SOLID		

ONE POINT LIQUID LIMIT FORMULA

$$LL = Wn \left(\frac{n}{25}\right)^{0.121}$$

WHERE:

LL = LIQUID LIMIT
Wn = MOISTURE CONTENT AT "n" BLOWS
n = THE NUMBER OF BLOWS ON THE LIQUID LIMIT CUP

ATTERBERG'S SOIL CONSISTENCY CLASSIFICATION SYSTEM

GENERAL MEMO

On March 29, 1972 a meeting was held between E. C. Hall and L. D. Suits from the Soils Laboratory, R. Cheney and R. Houghton from the Foundation Design Section, and R. Gemme from the Highway Design Section in an attempt to develop some guidelines to be used by the laboratory in presenting the results of the triaxial and consolidation tests in a uniform matter.

The results of the discussion are listed below:

Triaxial Test Presentation

CU Tests

a) Tests will still be run to 20% strain.

b) When drawing the Mohr Circles from the stress-strain curves, a value of stress will be chosen generally at 7% strain, unless there is a higher peak value at a lower strain.

c) The entire test, except for the Mohr Envelope and the values of "C" and "Ø", will be drawn in ink.

d) Tests which show questionable results will be presented in pencil.

CD Tests

a) Tests will be run to 20% strain.

b) The Mohr Circles will be drawn at the peak stress value, no matter what the value of strain is.

c) The Mohr Envelope will not be determined by the laboratory.

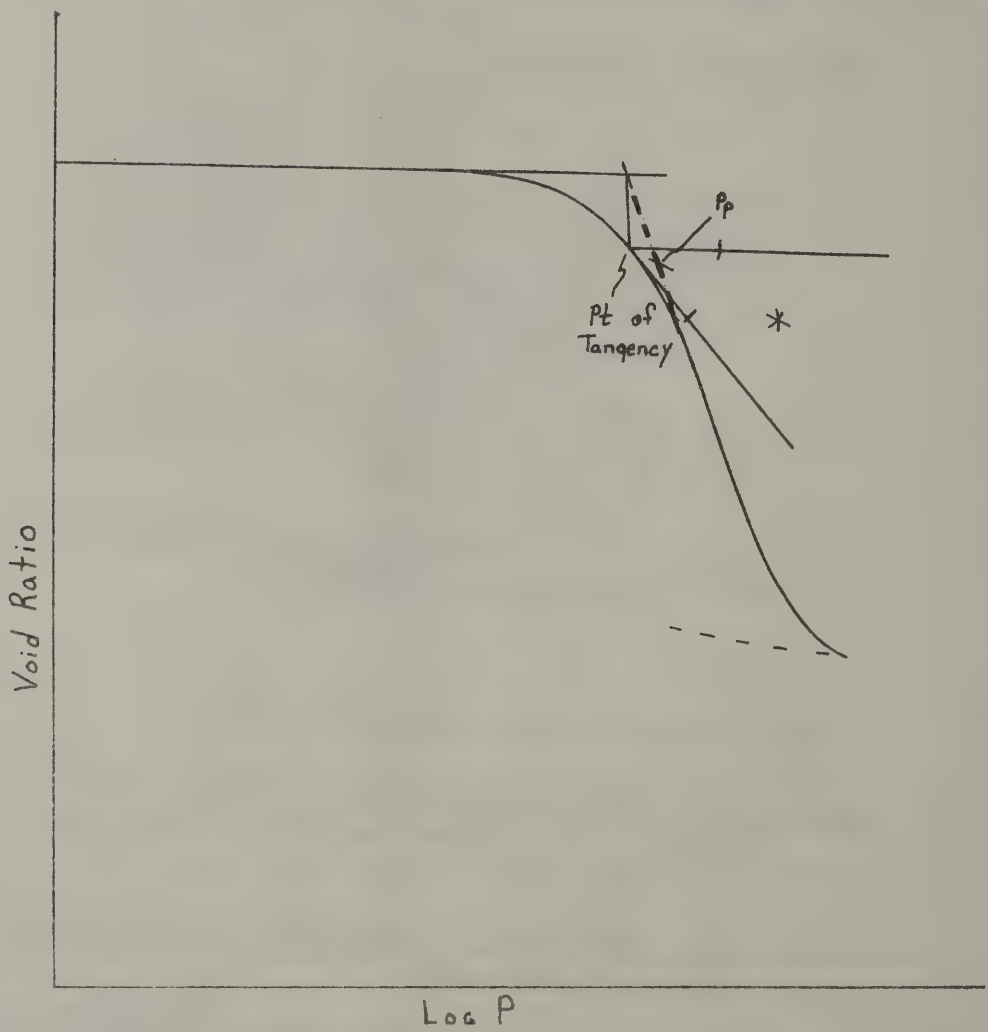
d) Except for above, the CD Test results will be presented as stated under CU Tests.

General Memo
March 31, 1972
Page 2

Consolidation Test Presentation

Preconsolidation Pressure Determination

a) Pp will be determined according to the diagram below. ①
Only one value of Pp will be determined. All drawing except construction lines will be in ink.
① *Except when minimum radius of curvature is beyond indicated point.*



b) On a recycle test the value of Pp will not be determined by the laboratory.

Coefficients of Consolidation

a) C_v

1) The Engineer will determine at what load readings will start to be taken for C_v. (USUALLY ONE LOAD BELOW P_o)

2) The readings will be taken no matter what degree of consolidation takes place.

b) C_s

1) The coefficient of secondary consolidation will be determined for all samples with a moisture content of 50% or over.

Test Termination

a) A consolidation test will be terminated once three points have been obtained which will give a straight line on the virgin portion of the e-log P plot, providing the machine capacity is not exceeded.

General Notes

a) Atterberg Limits and Hydrometer Tests will be requested on all triaxial tests.

b) Any notes which will help to explain the test results will be placed on the test.

c) If an engineer has requested to be present when a test is set up, he will be notified when the test is to be set up. However the technicians will not chase the engineer down or wait an unreasonable length of time for him to come to the laboratory, unless circumstances warrant.

d) When the design engineer is through with the test information and tube logs, he should return them to the laboratory to be filed.

e) After these few guidelines have been accepted, any alteration should be discussed with the laboratory supervisor.

f) Upon the installation of the TACT Consolidation System some of the consolidation guidelines may be altered.

Any suggestions on how to improve or add to these guidelines will be appreciated.

These methods will be instituted on tests that are performed starting April 3, 1972.

LDS:SAS

II Detailed Project Testing

A. General Test Request

1. Designer prepares laboratory testing request. After review by supervisor, request is submitted to laboratory. Request should include P.I.N. Undisturbed sample logs are returned to laboratory. Designer keeps test folder.

B. Consolidation Testing

1. Loading Increment Times

   In order to increase the rate of production the following loading time increments will be used when practical in the laboratory.

   (a) Three hour increments - silty clays and clays of medium plasticity and most organic soils.

   (b) Twenty-four hour increments - gelatinous organic soils and very plastic clays.

   (c) The following sysmbols will be used on the e log p curve to denote time of loading -

   - ⚠ three hour increment
   - ⊙ overnight increment
   - ⊡ weekend increment or period greater than 24 hours.

2. Coefficient of Consolidation (C_v)

   C_v values will be computed for all loads starting at one load below overburden pressure.

3. Maximum Loading

   Tests will be carried to sufficient loads to plot three points in a straight line above the maximum past pre-consolidation load.

4. Unloading and Reloading Cycles

   (a) When used -
   Cycles may be requested on projects where additional information is required for very accurate settlement predictions.

   (b) Procedure -
   An unloading cycle will start at one load beyond the maximum past pre-compression load and return to one load less than overburden pressure in normal decrements and increments.

5. Designers Responsibility

   (a) List loading time increments.

   (b) Specify if recycle is desired.

D. Strength Test Requests

1. Test Pressures

   List consolidating or confining pressures in P.S.I. for each test.

2. If other than standard, specify strain rate.

E. Routine classification tests such as Limits, Hydrometer and Specific Gravity are performed on testing material.

III <u>Comments to Designers</u>

A. Designers are encouraged to personally review jar samples in determining the boundaries of soil layers. Long tables are available in the basement for laying out samples for entire drill holes. Remember hours of stability and settlement analysis are worthless if incorrect assumptions are made concerning the engineering properties of each layer and the boundaries of each layer in vertical and horizontal dimensions.

B. Note that the laboratory has many projects to schedule and coordinate. Changes in procedures or special directions and comments on testing should be made through Everett Hall.

C. After project is complete, designer will return laboratory folder to laboratory and necessary data will be filed.

D. Subsurface Explorations Logs - Laboratory logs will not be sent to the Regions unless requested in an official manner.

E. Desginer should not alter test results on test forms.

Lyndon H. Moore

Lyndon H. Moore
Assistant Director

LHM/bs

THE USE OF X-RAYS IN SOIL TESTING

By: Everett C. Hall
Engineering Laboratory Supervisor
and
David Suits
Assistant Soils Engineer
Soil Mechanics Bureau
NYS Department of Transportation
Albany, New York

Introduction

Several months ago the Soil Mechanics Bureau of the N.Y.S.D.O.T. began taking x-rays of all Shelby tube samples, to obtain further information from the undisturbed soil samples. It was immediately seen that the x-ray was going to be a useful tool.

Learning to interpret the x-ray was done by extruding an entire tube sample, slicing it vertically and comparing it with the x-ray. The denser the material, the lighter it will show up on the x-ray. Stones are indicated by irregular very light, almost white, areas. Voids and organic material are the other extreme, appearing very dark or almost black. Silts and clays have been found to appear in varying shades of gray. Sand results in a salt and pepper appearance. The sand grains appear very light while the voids show up very dark.

Equipment

The equipment used is a Danish made Andrex unit belonging to the N.Y.S.D.O.T. Materials Bureau used for x-raying welds. It has a peak output of 300 k.v. and 6 milliamps. Through trial and error it was found that generally an output of 180 k.v. peak, 5 milliamps, and a 4 minute exposure provides the best contrast. At times it may be necessary to change these settings if the material to be x-rayed contains large amounts of organic material or marl. The tube to be x-rayed is placed 30 inches from the source.

The film used is Kodak Industrial Type AA, 4½ inches wide and 17 inches long. The per sheet cost of the film is between 45-50¢. The distance from the source to the Shelby tube allows all but about an 1 inch of material to be x-rayed, if the tube contains the maximum 18 inches of material.

Uses

To what use can an x-ray of an undisturbed soil sample be put? There are two general areas which are served by the x-rays. They are the establishment of a monitoring system of drilling procedures, and a visual aide to the design engineer in determining the existing soil profile and selecting strata for testing.

Drilling

Thus far the monitoring system of drilling procedures has probably been the largest benefit. This can best be shown by the following case history.

On one large project with over 150 undisturbed soil samples, the first twenty samples were forwarded to the Soil Mechanics Bureau and x-rayed immediately. The x-rays showed arched layers of material throughout the samples, see Fig. 1. This type of disturbance indicates overpressing of the sampler into the soil. After looking over the x-rays, the drill supervisors were able to return to the driller and correct his procedures before the first hole was completed. This saved time and money in that only the top half of the first hole had to be redrilled. The quality of drilling on the remainder of the project was improved.

The monitoring of drilling procedures includes visits to the drilling sites by the drill supervisor. Using a portable fluorescent lamp (Fig. 3), the x-rays can be shown to the drillers on site. In the case of contract drilling, x-rays will assist in the acceptance or rejection for payment of undisturbed soil samples.

When profiling the undisturbed sample tube the appearance of arched layers has always raised the question as to whether the arching was caused by improper drilling or the jacking procedure used for extruding samples in the lab. The x-ray shows that it occurs in drilling.

The appearance of several inches of wash material at the top of the tube in the x-ray indicates improper cleanout procedures are being followed during drilling (Figure 2). The presence of wash material has always been detectable upon opening the tube, but with the x-ray the ability to determine the amount ahead of time is the prime benefit.

Design

In design, the x-ray allows more selective testing to be done. Prior to using the x-ray technique the entire profile of the tube was not really known as only the top two to three inches was extruded for the purpose of visual identification. Now the presence of layering, a change in the material, old shear planes, stones, wood, etc. can be seen. See Figs. 4 and 5. In one instance a PCV valve was found (Fig. 6), while in another a small screw from the drilling equipment showed up on the x-ray.

Again a case history will better explain the use in design.

An undisturbed drill hole had been progressed in an area where there had been a shear failure. When the tubes from this hole were x-rayed, a shear plane was clearly visible. It was decided to determine the strength of the soil along the shear plane. As the x-ray is full scale, a measurement was made on the x-ray to determine the exact location of the plane in the tube. That amount of material was extruded, trimmed and tested.

Many times a layer of material which would have been vital in design has been missed because of not being found in the initial profiling of the tube sample. Therefore the initial testing may not have been on representative material. It is estimated that 10-20% of testing in the past has fallen into this category. Now the desired layer or combination of layers may be selected prior to opening the tube.

The presence of a small stone or object concealed in the sample, making trimming of the sample difficult or impossible, can be determined before trimming and can be avoided. Prior to the use of the x-ray, considerable time was wasted trying to trim samples with concealed pebbles.

Summary

The use of the x-ray has greatly increased the ability to monitor the procedures which are being used to obtain undisturbed soil samples. This allows correction of improper sampling procedures much sooner than previously, thus saving time and money in not having to redrill entire subsurface explorations. In the design process we now have the ability to be much more selective in testing, to have an overall picture of the soil profile, and to save time by not testing material that is not representative of the soil deposit.

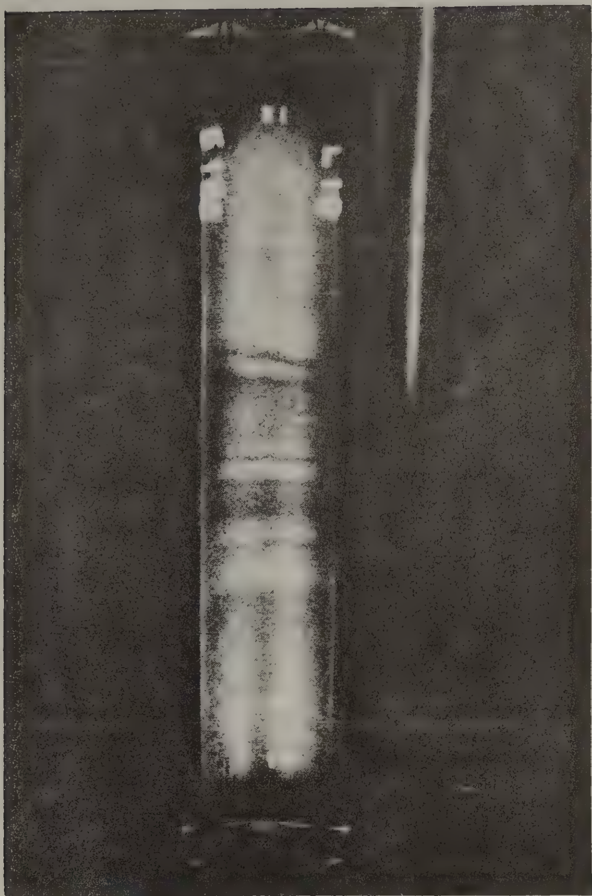


Fig. 1
Arched Layering

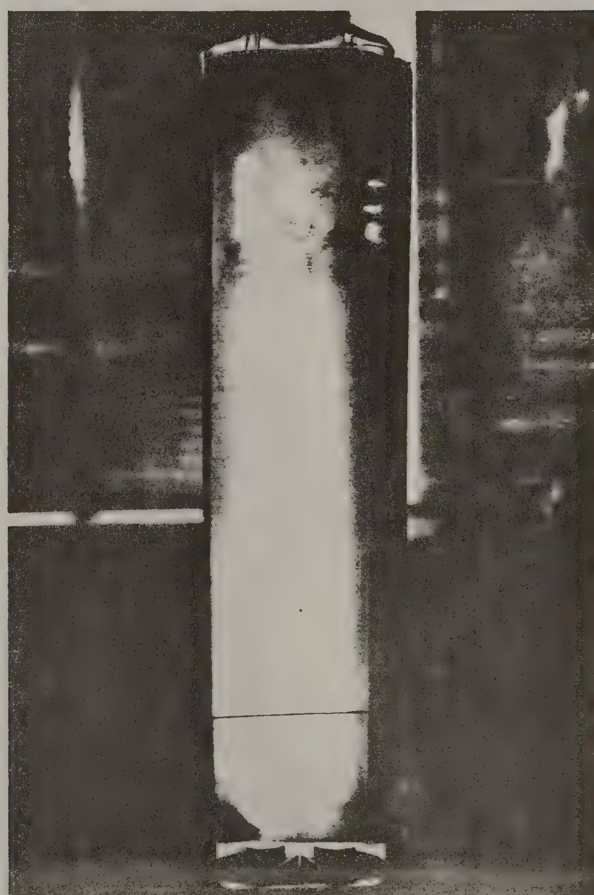


Fig. 2
Wash Material

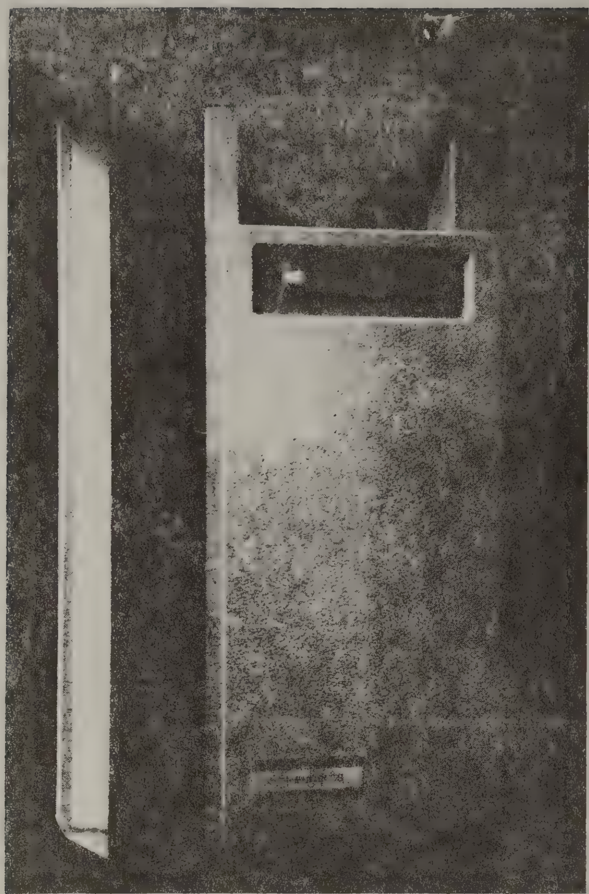


Fig. 3
Fluorescent lamp used
for on site viewing

Fig. 4
Undisturbed layered
sample.

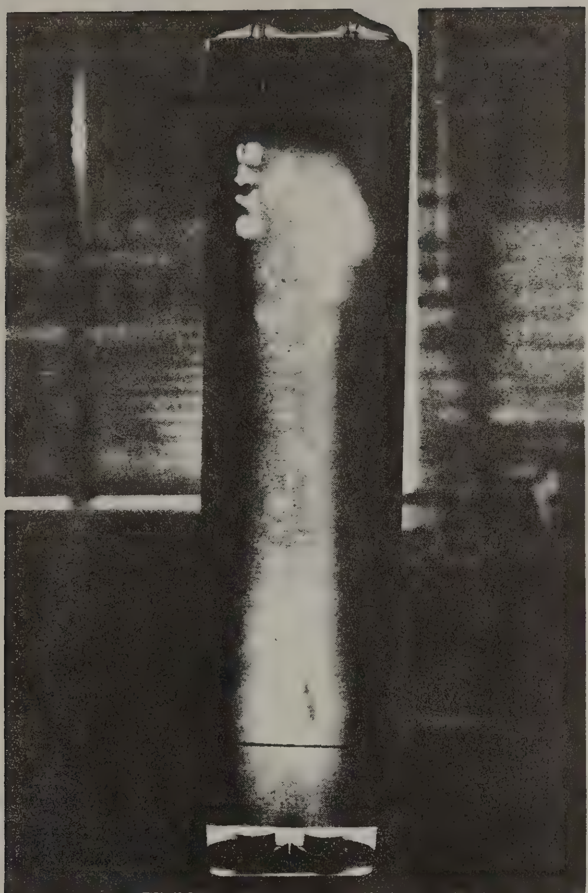
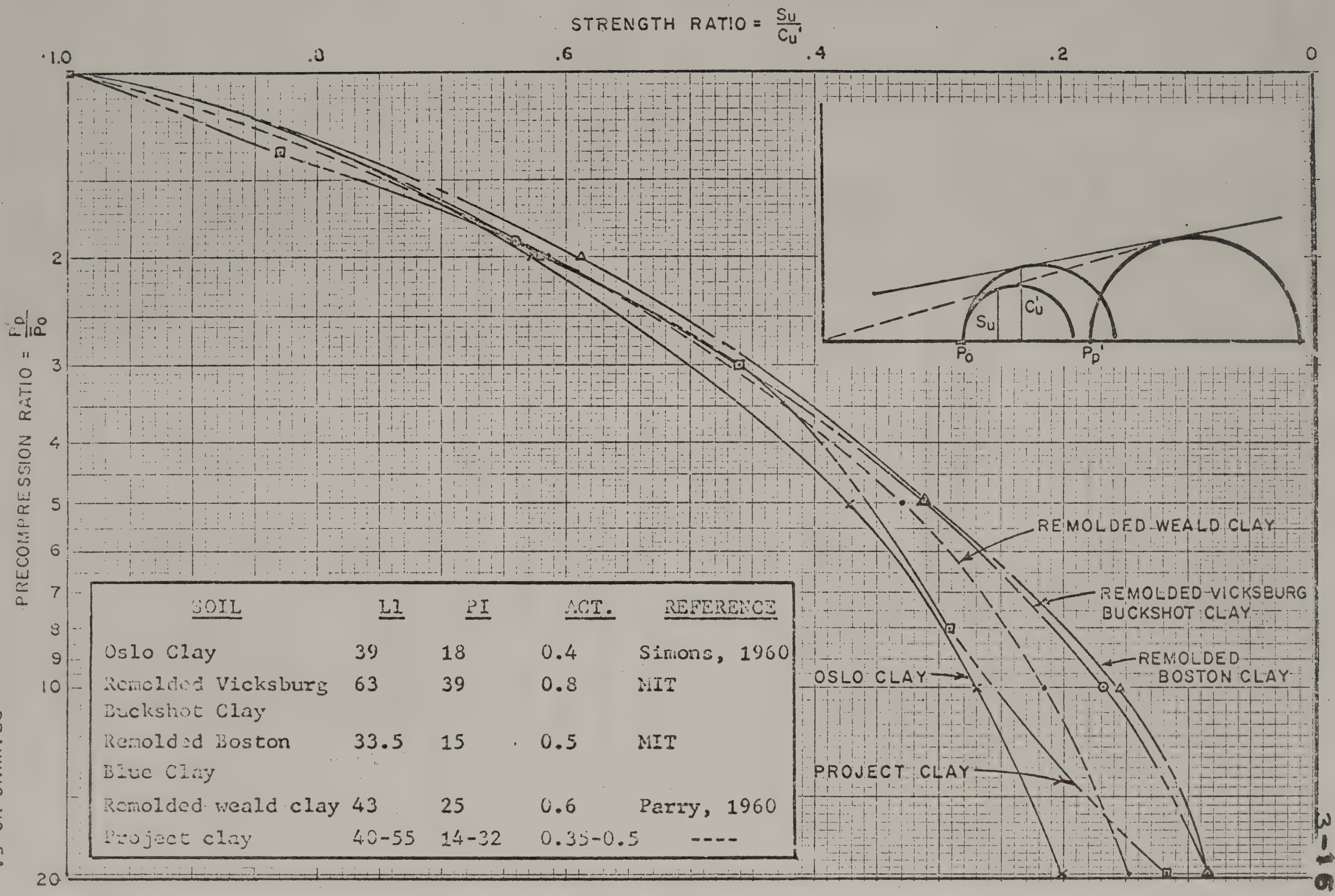


Fig. 5
Stones located
throughout sample.

Fig. 6
PCV valve located
at top of sample.

EFFECT OF PRECOMPRESSION ON UNDRAINED SHEAR STRENGTH



SOIL	LL	PI	ACT.	REFERENCE
Oslo Clay	39	18	0.4	Simons, 1960
Remolded Vicksburg Buckshot Clay	63	39	0.8	MIT
Remolded Boston Blue Clay	33.5	15	0.5	MIT
Remolded weald clay	43	25	0.6	Parry, 1960
Project clay	40-55	14-32	0.35-0.5	----

DRAWING NO. 5A

Shearing Strengths of Kaolinite, Illite, and Montmorillonite[a]

Closure by Roy E. Olson,[3] F. ASCE

Chowdhury notes that the definition of $\bar{\phi}_R$ in Appendix II of the paper is inconsistent with the use of the same term in the text. The definition of Appendix II is in error. The writer uses $\bar{\phi}_R$ to denote the effective angle of internal friction at failure in an $\bar{R}$ test and $\bar{\phi}_r$ to denote the effective residual angle of internal friction.

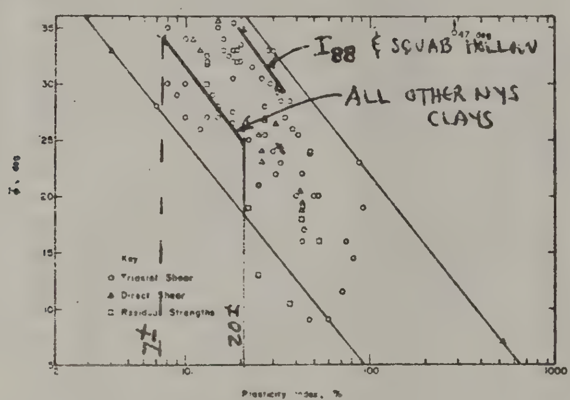


FIG. 12.—Influence of Plasticity Index on Effective Angle of Internal Friction as Determined Using Both Drained and Undrained Tests

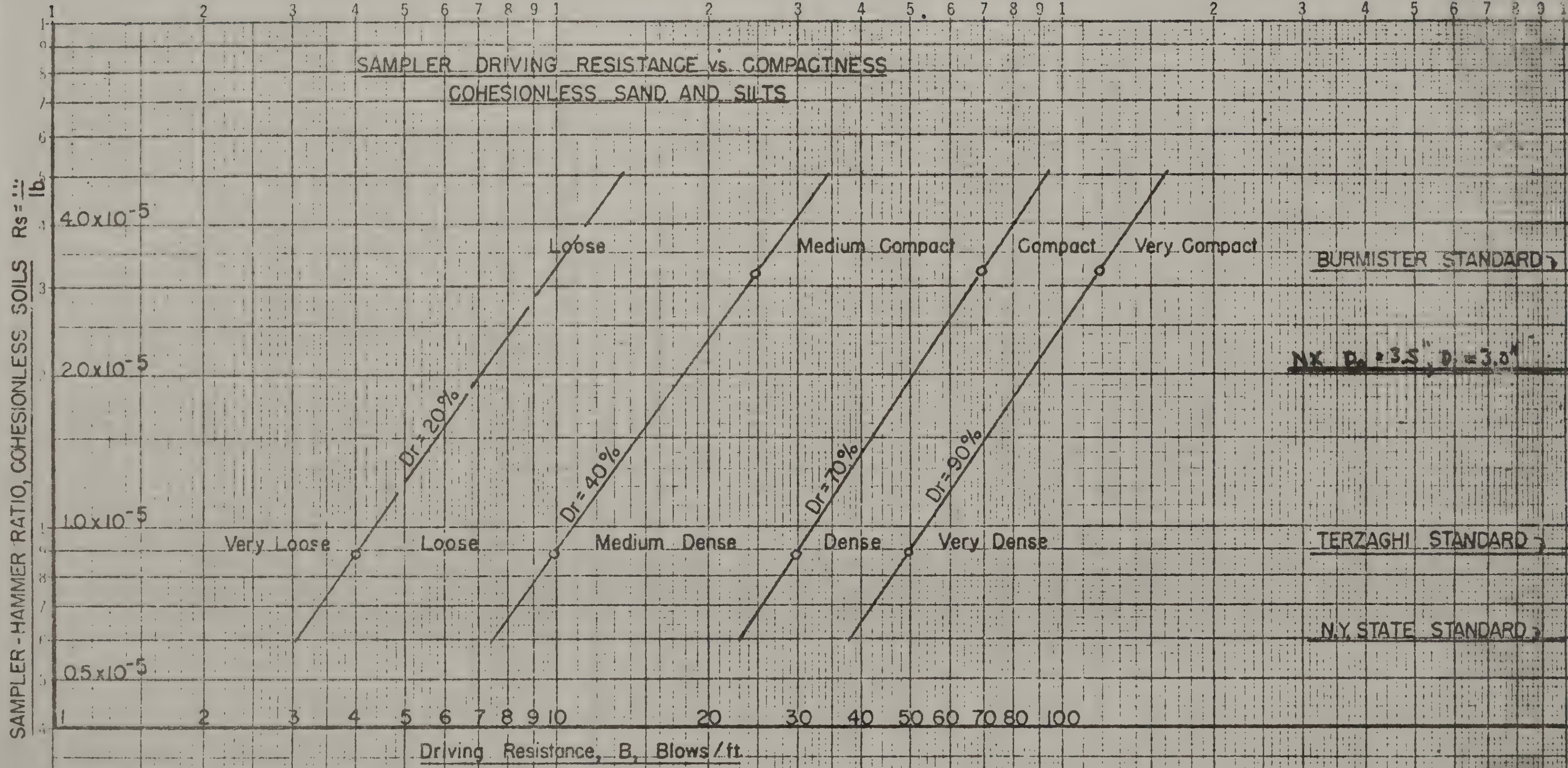
Chowdhury references a curve of $\bar{\phi}$ versus plasticity index presented by Terzaghi and Peck (34, p. 112), suggests the curve applies to natural soils, and then attempts to compare values of $\bar{\phi}$ from the writer's paper with values derived from Terzaghi and Peck's curve, but without presenting a figure nor numerical values. It should be noted that: (1) Terzaghi and Peck's curve was intended to apply to both remolded and undisturbed soils; (2) Terzaghi and Peck neglect to present any data in their plot so the reader cannot determine the number of observations nor the amount of scatter; and (3) Terzaghi and Peck restrict the relationship to "clays of moderate to low sensitivity under drained conditions." In 1966 the writer tried to correlate $\bar{\phi}$ with plasticity index using all the data that could be found in the literature at that time. The resulting correlation is shown in Fig. 12. Considering the scatter in the data it is difficult to understand how any definitive conclusions can be drawn from such a

[a] November, 1974, by Roy E. Olson (Proc. Paper 10947).

[3] Prof. of Civ. Engrg., Dept. of Civ. Engrg. Univ. of Texas, Austin, Tex.

SECTION 4

FIELD TESTS AND MEASUREMENTS



STANDARD	W(lb.)	H(in.)	Do(in.)	Di(in.)	Rs (ft²/lb)
BURMISTER	250	20	3.625	2.930	3.12×10^{-5}
TERZAGHI	140	30	2.000	1.375	0.895×10^{-5}
N.Y. STATE	300	18	2.000	1.500	0.594×10^{-5}

W = Weight of Hammer, lb.; H = Height of Drop, in.

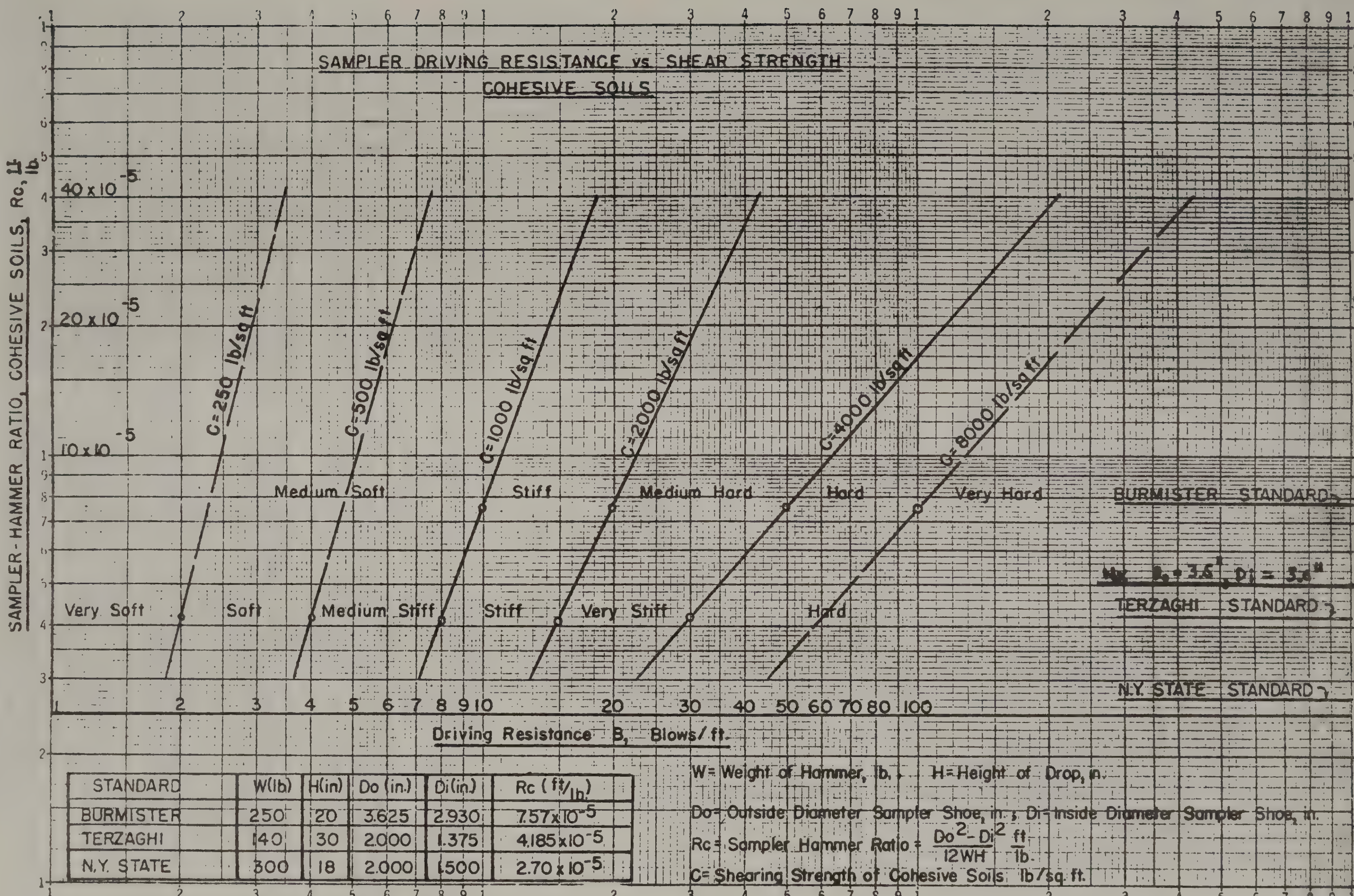
Do = Outside Diameter Sampler Shoe, in.; Di = Inside Diameter Sampler Shoe, in.

$$R_s = \text{Sampler Hammer Ratio} = \frac{Do^3 - Di^3}{144WH} \ \frac{ft^2}{lb}$$

Dr = Relative Density, %

N.Y.S.-D.P.W. BUREAU OF SOIL MECHANICS

COMPILED BY: B.E.B. FEB. 20, 1958



STANDARD	W(lb)	H(in)	Do (in.)	Di(in.)	Rc (ft/lb)
BURMISTER	250	20	3.625	2.930	7.57×10^{-5}
TERZAGHI	140	30	2.000	1.375	4.185×10^{-5}
N.Y. STATE	300	18	2.000	1.500	2.70×10^{-5}

W= Weight of Hammer, lb.; H=Height of Drop, in.

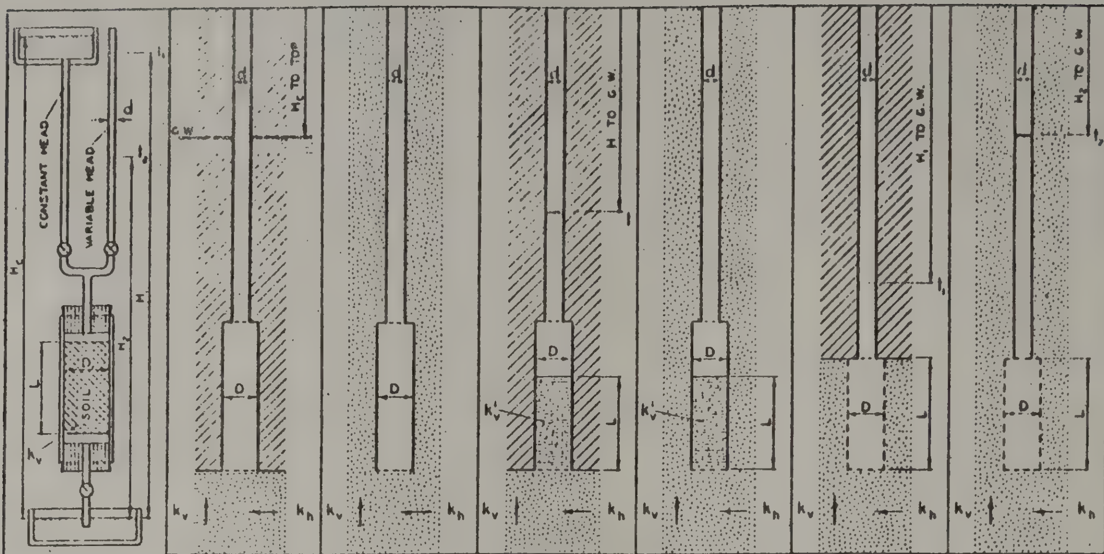
Do= Outside Diameter Sampler Shoe, in.; Di= Inside Diameter Sampler Shoe, in.

Rc= Sampler Hammer Ratio = $\frac{Do^2 - Di^2}{12WH}$ $\frac{ft}{lb}$.

C= Shearing Strength of Cohesive Soils lb/sq. ft.

N.Y.S.-D.P.W. BUREAU OF SOIL MECHANICS

COMPILED BY: B.E.B. FEB. 20, 1958



LABORATORY PERMEAMETER (CONSOLIDOMETER)	FLUSH BOTTOM AT IMPERVIOUS BOUNDARY	FLUSH BOTTOM IN UNIFORM SOIL	SOIL IN CASING AT IMPERVIOUS BOUNDARY	SOIL IN CASING IN UNIFORM SOIL	WELL POINT-FILTER AT IMPERVIOUS BOUNDARY	WELL POINT-FILTER IN UNIFORM SOIL
A	B	C	D	E	F	G

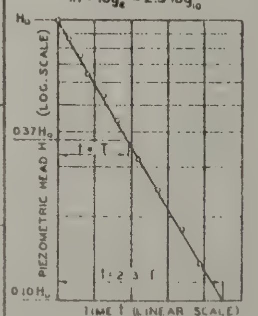
CASE	CONSTANT HEAD	VARIABLE HEAD	BASIC TIME LAG
A	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 \cdot H_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ FOR $d = D$
B	$k_m = \frac{q}{2 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8 (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ FOR $d = D$
C	$k_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11 (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $d = D$	$h_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ FOR $d = D$
D	$k'_v = \frac{4 \cdot q \cdot \left[\frac{\pi}{8} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2 H_c}$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $k'_v = k_v$, $d = D$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m}\right] + L}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ FOR $k'_v = k_v$, $d = D$
E	$k'_v = \frac{4 \cdot q \left[\frac{\pi}{11} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{\pi D^2 H_c}$	$k'_v = \frac{d^2 \left[\frac{\pi}{11} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $k'_v = k_v$, $d = D$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot T}$ $k'_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ FOR $k'_v = k_v$, $d = D$
F	$k_h = \frac{q \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \pi L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{2mL}{D} > 4$
G	$k_h = \frac{q \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \pi L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{mL}{D} > 4$

NOTATION

D = DIAM, INTAKE, SAMPLE, CM
d = DIAMETER, STANDPIPE, CM
L = LENGTH, INTAKE, SAMPLE, CM
H_c = CONSTANT PIEZ. HEAD, CM
H_1 = PIEZ. HEAD FOR $t = t_1$, CM
H_2 = PIEZ. HEAD FOR $t = t_2$, CM
q = FLOW OF WATER, CM^3/SEC.
t = TIME, SEC.
T = BASIC TIME LAG, SEC.
k'_v = VERT. PERM. CASING, CM/SEC.
k_v = VERT. PERM. GROUND, CM/SEC.
k_h = HORZ. PERM. GROUND, CM/SEC.
k_m = MEAN COEFF. PERM, CM/SEC.
m = TRANSFORMATION RATIO

$k_m = \sqrt{k_h \cdot k_v}$ $\quad m = \sqrt{k_h / k_v}$

$\ln = \log_e = 2.3 \log_{10}$



DETERMINATION BASIC TIME LAG T

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v AND k_h CONSTANT) - NO DISTURBANCE, SEGREGATION, SWELLING OR CONSOLIDATION OF SOIL - NO SEDIMENTATION OR LEAKAGE - NO AIR OR GAS IN SOIL, WELL POINT, OR PIPE - HYDRAULIC LOSSES IN PIPES, WELL POINT OR FILTER NEGLIGIBLE

Fig. 18. Formulas for determination of permeability

REF: "TIME LAG AND SOIL PERMEABILITY IN GROUND-WATER OBSERVATIONS" BULLETIN NO. 36, CORPS OF ENGINEERS WATERWAYS EXPERIMENT STATION.

FIELD VANE SHEAR TEST PROCEDURE

1. Drive casing to depth desired and positively clamp casing to rig to prevent settlement and rotation prior to vane testing.

2. Clean out casing by jet augering to bottom of casing.

3. Attach vane, vane rods and rod bearing guides to drill rods and lower bottom of assembly to within one to three feet above bottom of casing. Positively hold drill rods and assembly at this depth with pipe clamp. Rod bearing guides shall be attached at every 30 feet of depth. All connections shall be wrench tight.

4. Lower casing adapter and Acker vane torque head equipment over top of rods to rest on pipe clamp.

5. Attach and hold drill rods with hoisting plug.

6. Remove pipe clamp and attach casing adapter (wrench tight) to casing and vane equipment to adapter.

7. Lower bottom of vane into soil below bottom of casing to proper depth (12 inches plus length of vane i.e. 18 inches below bottom of casing for 6 inch long vane). Place alignment line on rods and reference point on Acker head prior to lowering assembly to insure vane enters soil with no rotation.
   Note: If resistance when pressing vane to desired elevation is high (greater than weight of one man) the vane test should be attempted at a deeper depth (maximum 5 feet more) until softer material is encountered. If resistance when pressing vane is very irregular it should be noted on vane data recording sheet.

8. Lock torque arm to rods with set screws and slacken hoisting plug rope. (Check alignment between torque arm and pressure gauge and between alignment line on rods and reference point on Acker head).

9. Start first determination of vane test by turning crank with uniform, continuous 12 revolutions per minute. (This is equivalent to 6^{o}/min. rotation of the vane on the Acker assembly or 1.67% strain/min.) or at a rate ordered by the engineer.

10. Record time, crank turns and force gauge readings until the peak force is reached (usually 5 to 10 minutes but not less than 2.5 minutes for this determination). Run one test in every third test with depth well beyond the peak force (usually 15 to 20 minutes).

11. While holding rods with hoisting plug, loosen screws on torque arm and lower vane an additional foot into the soil and repeat steps 8, 9 and 10 for second determination of vane test.

12. Run one rod drag test at every third test with depth or when ordered by the engineer by repeating steps 1 through 10 without the vane on the end of the vane rods.
    Note: The bottom of the vane rod should be lowered to the same depth as it existed for the second vane determination test.

13. Obtain a sample of the soil within the depth of the vane test (the depth of the two vane determinations) by attaching a spoon extension to the 18 inch spoon and sampling to the bottom elevation of the second vane determination. Split the samples into two parts A and B. Part A would represent the first vane determination and Part B the second vane determination.

General

1. Keep hole filled with water at all times for soft soils.
2. If there is run-in at bottom of hole, leave last 2 to 4 in. plug in bottom of casing.
3. Stop vane tests before exceeding capacity of force gage.
4. Check alignment of vane rod and drill rod connection periodically by rotating assembly on flat surface.

FIELD VANE CONVERSION FACTORS

H = HEIGHT OF VANE IN INCHES

D = DIAMETER OF VANE IN INCHES

T = TORQUE ON VANE PRODUCED BY FORCE ON LEVER ARM = FL

F = FORCE ON VANE LEVER ARM IN POUNDS

L = MOMENT ARM LENGTH WHERE FORCE IS APPLIED IN INCHES

SS = SHEAR STRENGTH OF SOIL = $864T/\pi D^2(3H+D)$ = KF IN POUNDS/FEET^2

K = FORCE TO SHEAR STRENGTH CONVERSION FACTOR = $864L/\pi D^2(3H+D)$ IN FEET^{-2}

FOR H/D = 2 AND L = 6 IN.

D x H	$K = \frac{236}{D^3 \text{ FEET}^2}$
2" x 4"	29.5/FEET^2
$2\frac{1}{2}$" x 5"	15.1/FEET^2
3" x 6"	8.74/FEET^2

FOR H/D = 4 AND L = 6 IN.

D x H	$K = \frac{127}{D^3 \text{ FEET}^2}$
2" x 8"	15.9/FEET^2

NOTES: 1. CONVERSION FACTORS (K) VARY PROPORTIONATELY WITH CHANGES IN MOMENT ARM (L).

2. SHEAR STRENGTH (SS) VERSUS FORCE (F) TABLES ARE ATTATCHED FOR THE ABOVE CONDITIONS OF D X H AND L = 6 IN.

L (MOMENT ARM) = 6 INCHES

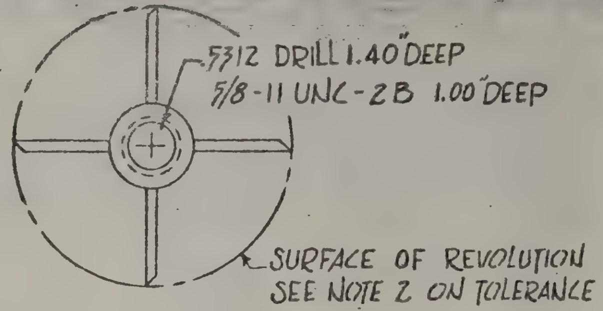
F (POUNDS)	SS (SHEAR STRENGTH) IN POUNDS/FEET^2 FOR FOLLOWING D x H			
	2" x 4"	$2\frac{1}{2}$" x 5"	3" x 6"	2" x 8"
1	29.5	15.1	8.74	15.9
2	59.0	30.2	17.5	31.8
3	88.5	45.3	26.2	47.7
4	118	60.4	35.0	63.6
5	148	75.5	43.7	79.5
6	177	90.6	52.4	95.4
7	207	106	61.2	111
8	236	121	69.9	127
9	266	136	78.7	143
10	295	151	87.4	159
11	325	166	96.1	175
12	354	181	105	191
13	384	196	114	207
14	413	211	122	223
15	443	227	131	239
16	472	242	140	254
17	502	257	149	270
18	531	272	157	286
19	561	287	166	302
20	590	302	175	318
21	620	317	184	334
22	649	332	192	350
23	679	347	201	366
24	708	362	210	382
25	738	378	219	398
26	767	393	227	413
27	797	408	236	429
28	826	423	245	445
29	856	438	253	461
30	885	453	262	477
31	915	468	271	493
32	944	483	280	509
33	974	498	288	525
34	1003	513	297	541
35	1033	529	306	557
36	1062	544	315	572
37	1092	559	323	588
38	1121	574	332	604
39	1151	589	341	620
40	1180	604	350	636
41	1210	619	358	652

L (moment Arm) = 6 inches

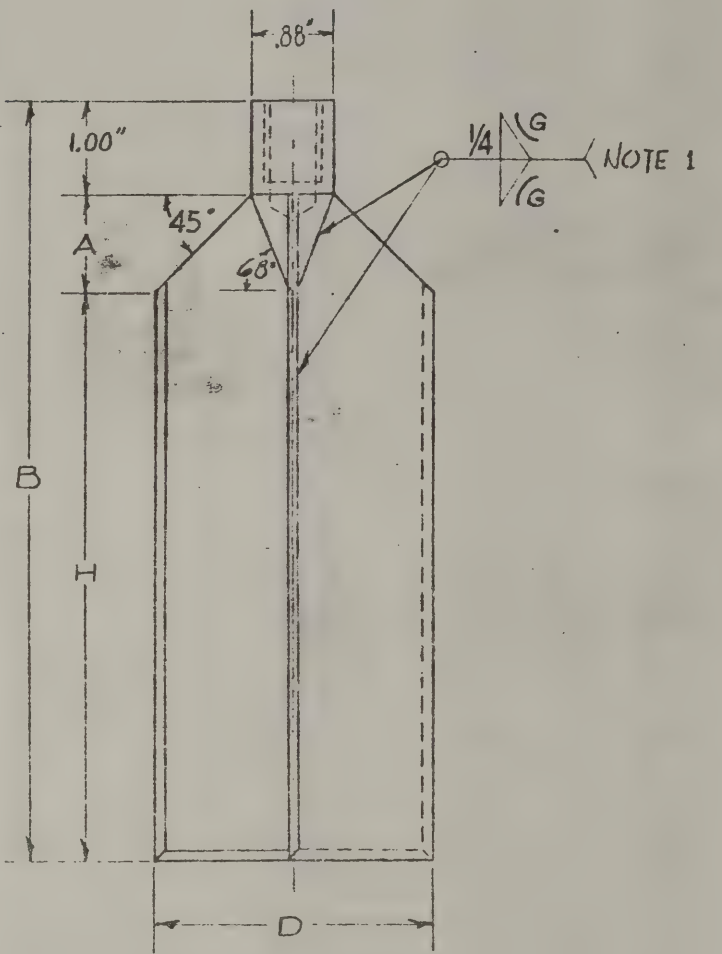
SS (SHEAR STRENGTH) in POUNDS/FEET^2 FOR FOLLOWING D×H

F (Pounds)	2" x 4"	2½" x 5"	3" x 6"	2" x 8"
42	1239	634	367	668
43	1269	649	376	684
44	1298	664	385	700
45	1328	680	393	716
46	1357	695	402	731
47	1387	710	411	747
48	1416	725	420	763
49	1446	740	428	779
50	1475	755	437	795
51	1505	770	446	811
52	1534	785	454	827
53	1564	800	463	843
54	1593	815	472	859
55	1623	831	481	875
56	1652	846	489	890
57	1682	861	498	906
58	1711	876	507	922
59	1741	891	516	938
60	1770	906	524	954
61	1800	921	533	970
62	1829	936	542	986
63	1859	951	551	1002
64	1888	966	559	1018
65	1918	982	568	1034
66	1947	997	577	1049
67	1977	1012	586	1065
68	2006	1027	594	1081
69	2036	1042	603	1097
70	2065	1057	612	1113
71	2095	1072	621	1129
72	2124	1087	629	1145
73	2154	1102	638	1161
74	2183	1117	647	1177
75	2213	1133	656	1193
76	2242	1148	664	1208
77	2272	1163	673	1224
78	2301	1178	682	1240
79	2331	1193	690	1256
80	2360	1208	699	1272
81	2390	1223	708	1288
82	2419	1238	717	1304
83	2449	1253	725	1320
84	2478	1268	734	1336
85	2508	1284	743	1352
86	2537	1299	752	1367

MATERIAL SPECIFICATIONS
VANES - TYPE 301 STAINLESS STEEL SHEET, 17 GA.
HUB - TYPE 316 STAINLESS STEEL COLD DRAWN ROD.



DIMENSION	VANE 1	VANE 2	VANE 3	VANE 4
D	2"	2½"	3"	2"
H	4"	5"	6"	8"
A	.56"	.81"	1.06"	.56"
B	5.6"	6.8"	8.1"	9.6"



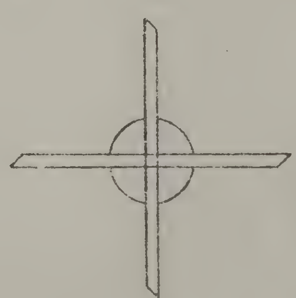
NOTES

1. FILLET WELDS SHALL BE MACHINE GROUND TO A UNIFORM CONCAVE RADIUS OF .25" BY MEANS OF AN APPROPRIATE WHEEL. FINISHED WELDS ARE TO BE FREE OF ALL DEFECTS.

2. FINAL FINISHED UNITS SHALL DESCRIBE A CONCENTRIC CYLINDRICAL SURFACE OF REVOLUTION WITH A DIAMETRAL TOLERANCE OF .05" TOTAL WHEN ROTATED IN THE AXIS OF THE HUB THREAD CONNECTION.

REVISIONS

1	REMOVED BEVEL AREA "A"	1/18/62	J.H.
2			
3			
4			



NOT TO SCALE

FIELD VANE SHEAR APPARATUS

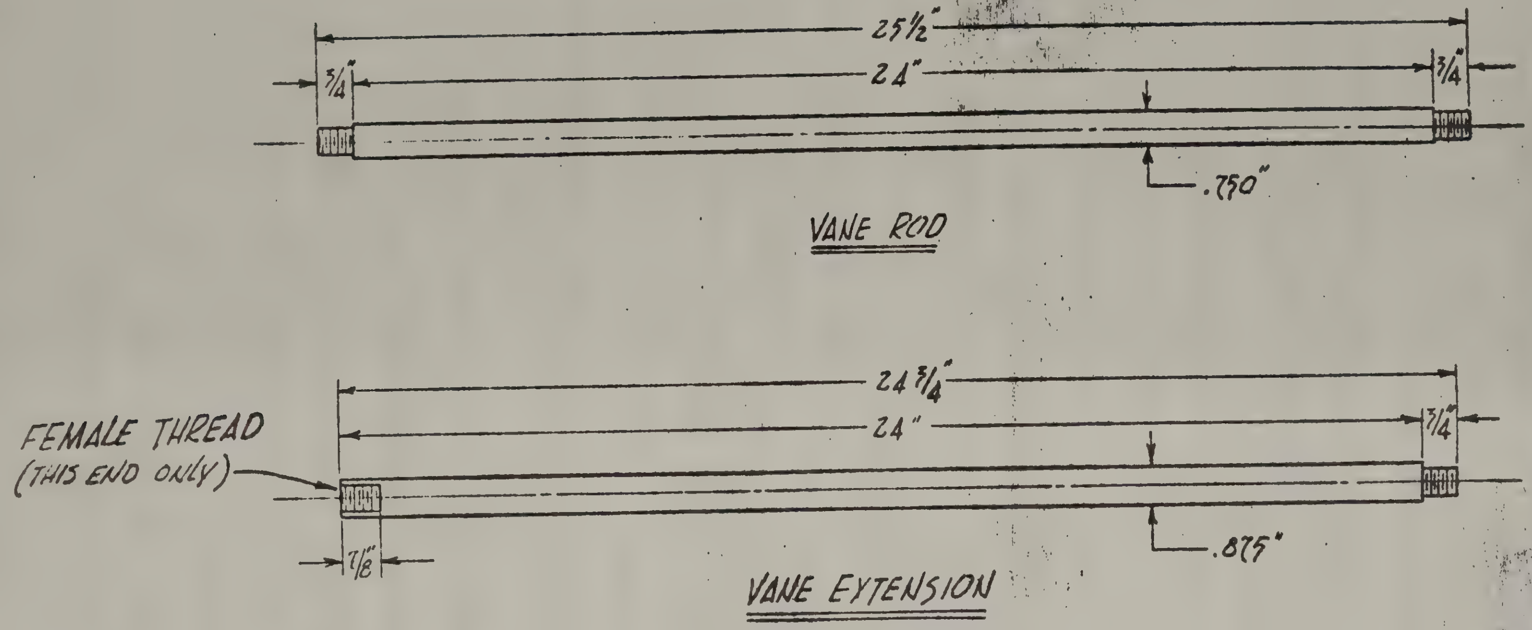
VANE DETAILS

9/26/61

D.J.B.

Wm. P. Hofmann

SM 1604 R1



NOTES: 1. ALL THREADS TO BE ⅝ x 11 N.C.
2. MATERIAL TO BE TYPE 316 STAINLESS STEEL COLD, DRAWN ROD.

SCALE: ¼" = 1"

	REVISIONS		
1	Added - Female Thread (This end only)	3/26/62	D.J.B.
2			
3			
4			

DRAWN BY J.H. D.J.B. J.H.

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

FIELD VANE SHEAR ROD
AND
EXTENSION ROD

APPROVED Wm. P. Hofmann

DATE JAN. 22, 1962

SM 1611R1

SLOPE INDICATOR INSTALLATION

Setup

If water is a problem, recirculating should be considered, because a tank is needed to mix grout in any case. Recirculating also eliminates the need of shutting down a traffic lane with a truck and water tank. The rig should be fitted with a 10 ft. "NW" rod in the spindel. The rig cable should be reaved over the crown sheave in a hoisting position. It will be useful during the installation.

Equipment Needed

The following is a minimum list of tools and equipment needed to install a Slope Indicator. Any attempt to "do without" or substitute needed tools or equipment is not economical and is time consuming. Drill trucks have ample space to haul all equipment necessary to complete any installation.

NOTE: * indicates a sufficient quantity to complete the boring.

1. * "HW" casing
2. * "HW" Drive shoe
3. 2 "HW" Drive heads (complete)
4. * "NW" and "AW" Drill rod
5. 2 - 5 ft. sections of "NW" drill rod
6. 2 - 2 ft. sections of "NW" drill rod
7. 1 each "NW" open and closed rod ring
8. 1 each "NW" open and closed rod lifter
9. 1 - plain type water swivel
10. 1 - 3-3/4" ROCK ROLLER BIT if 3-1/2" indicator casing is used and 1 5-5/8 roller bit
11. 1 - 3-1/2" Chopping bit
12. 1 - 5' "NX" Double tube core barrel and bit
13. * Bentonite (QUICK GEL)
14. * Cement
15. 2 - 5' lengths of 6" pipe
16. 1 - 6" drive head with coupling attached
17. 1 - 3-1/2" split barrel sampler with trap

18. 1 drive head for "NW" rods
19. * Slope Indicator Casing (3-1/2 or 2-7/8) with couplings for each
20. Check Valve Assembly
    - A. Slope Indicator cap with 1" hole
    - B. 2 - 1" close nipple
    - C. 1 - 8" length of 1" pipe (with 12 - 3/8" holes)
    - D. 1 - 1" pipe cap
    - E. 1 - 1" pipe coupling
    - F. 1 - 1" check valve
21. 1 - 6" protective cap (furnished by Bureau)
22. * 1" rigid plastic pipe in 10 ft. lengths with coupling for each
23. 2 - female 1" slip fit to 1" pipe adaptors
24. 2 - snap on to 1" male pipe adaptor
25. 1 - can glue (PVC)
26. 1 - hand drill
27. 1 - alignment tool for slope indicator pipe used
28. 20' of 1/4" manila rope
29. 1 - 5 gal. pail
30. 1 - 1 gal. pail
31. pop rivet gun and rivets (furnished by Bureau)
32. 2 - 3" round bristle brushes (furnished by Bureau)
33. 1 - Master #17 lock (furnished by Bureau)
34. Centrifugal pump (2" size)

Procedure

After the rig is set up and all equipment needed is at the site, the hole is started. Six inch casing is driven at least five ft., but no more than ten ft., except in special cases or when directed by the engineer. The six inch casing is cleaned to the bottom using the 5-5/8" roller bit.

Samples should be taken if requested by the engineer. If no samples are requested the hole can be advanced in a standard method without sampling.

Using the "HW" casing the hole should be advanced to ledge or in some cases till.

If ledge is encountered, an "NX" core should be run no less than five ft. This core is used to verify that the bottom of the Indicator will be anchored at this elevation when grouted. If 3-1/2" indicator casing is to be used a 3-3/4" roller bit must be used to ream out the hole so it will accept the casing and allow room for the grout to get by.

If till is used to anchor the casing, no core is taken but a driven sample should be tried using a 3-1/2" sampler. This will be used to verify the till layer. Then the hole is drilled no less than five ft. with the 3-3/4" roller bit and washed clean.

The "HW" is seated on ledge or till and a five ft. hole is open ahead of it. At this point the hole should be ready for the Slope Indicator casing to be installed.

The check valve assembly is fitted to a ten ft. length of indicator casing using at least two pop rivets. Tape is wrapped around the casing where it meets the cap and also over the holes in the rivets to stop grout from entering the casing. The casing is then lowered into the hole. The next 10 ft. length is fitted with a coupling and four pop rivets inserted. A 1/4" line is attached to the alignment tool and the line is threaded thru this 10 ft. length from the coupling end. The alignment tool is entered into the grooves of the S.I. pipe and left to protrude half of its length. This protruding half is entered into the grooves of the lower piece of pipe and the lower pipe is stabbed into the coupling. Pop rivets are inserted and this joint is taped. The alignment tool can be pulled thru the upper length of casing after it has been entered into the hole. The pop rivets <u>must</u> be placed between the grooves in the S.I. casing. All joints are taped to stop entry of grout.

As the casing enters the water filled hole it will become buoyant. In order to counteract this buoyancy, water is poured into the slope indicator casing. As it sinks a piece of 1/4" line is used to hold it or a chain vice if one is available. When the bottom of the hole is encountered a measurement should be taken to be sure it is at the desired elevation. The Slope Indicator casing is cut off at least one foot above the six inch pipe and two of the slots must be oriented perpendicular to the slide. In any case it must be cut off well above the "HW" casing so that the grout will not run into the S.I. casing when grouting starts. At this point the S.I. pipe must be tied down to the six inch pipe because the grout will float it out.

The next step is to install the grout pipe. A ten foot length of one inch rigid plastic pipe is fitted (with glue) to a slip-fit to pipe thread adaptor. Each successive ten foot length is fitted with a coupling and glued to the next. When the bottom of the S.I. casing is reached the adaptor is screwed onto the protruding one inch nipple. The top of the grout pipe is cut off at a convenient working height (about five ft. from the ground surface) and this end is also fitted with a slip-fit to one inch pipe thread adaptor. A snap on to one inch male fitting is screwed into the adaptor and it is ready to grout.

If sludge of any kind was left in the bottom of the hole from the drilling procedures water can be injected at this point to wash the S.I. casing down to the desired elevation.

The grout is then mixed using mix and volume charts available. The grout is mixed and delivered with a two inch centrifugal pump. The most efficient way to mix the grout is to mix the cement and water first and then add the quantity of Bentonite. After the proper amount of grout is mixed it can be pumped down the grout pipe. As the grout fills the hole the water will be expelled. When the grout flows from the "HW" casing, the grout is shut off but left to circulate in the tank.

A quantity of clear water is pumped down the grout pipe. This quantity should be one gallon for every twenty ft. of grout pipe. This will expell most of the grout from the grout pipe so that it will not fall back into the S.I. pipe when the grout pipe is removed. If no water surges back up the one inch grout pipe this is a good indication the check valve is holding. The grout pipe is then removed cutting each ten foot length with a hacksaw about four tenths below each coupling so the pipe can be reused again.

In order to insure that all the grout is out of the S.I. casing, a hose should be fitted with two three inch brushes for 3-1/2" installations and one 3" brush for 2-7/8" installations. This is done by taping the brushes to a hose that has no coupling on one end and entering it to the bottom of the S.I. casing. Water is pumped down and the hole is surged by hand until the water returns clear.

After all the grout pipe is out, and the S.I. casing has been washed out clean, a string of "AW" drill rod is lowered to the bottom. This will hold the S.I. casing from floating while the "HW" casing is being pulled. A plastic bag should be taped around the top of the S.I. casing to insure that no grout falls into it. The alignment of the grooves should be checked again to be sure it did not move.

The next step is to bump back the "HW" casing, refilling with grout from the surface after each five foot section is pulled. As soon as possible the "HW" casing should be pulled with the winch. From the time the grout is mixed to the point where all the "HW" casing is out of the ground should be one continuous operation. If any casing is left in overnight with grout in it, the grout will adhere to the S.I. casing and the two casings will be locked together.

After the grout has been let to partially set (about 15 hrs.) the six inch hole protector is screwed onto the six inch pipe and the S.I. casing is cut off flush with the top of the open protector. A pop rivet is installed in the wall of the S.I. casing for a reference point for elevation.

The last thing to be done is to measure the inside depth of the S.I. casing exactly. This measurement is recorded. The protective cap is then closed and locked with a master #17 lock supplied by the Bureau. The reason for using only this lock supplied by the Bureau is that they are all keyed alike and the engineering force can open any installation to read it with one key.

Slope Indicator Record Keeping Procedures
Roadway Foundation Section
12/13/77

S.I. records should include the following:

a). Plan + accurate survey (Hor. & Vert.) of top of casing.

b). X-sect. + identification of the problem (soil & water profile).

c). S.I. installation log.

d). Plot of depth vs. deflection at reading date.

e). Plot of time vs. deflection at depths of major movement.

f). Periodic written evaluation of installation with future instruction on maintenance and readings. Minimum one report per year.

Data should be handled in the following manner unless otherwise instructed:

a). Record data on form #SM 422 at two ft. intervals starting at the nearest footmark above the bottom unless this will place a wheel within 6" of a casing joint. The depth should be measured from the top of the cable guide and notes should identify if a casing extender is used.

b). The data should be reviewed and each reading compared to the previous reading. If any number changes by more than 20$\pm$ units, compute all deflections and plot as above.

c). The records shall be bound or attached in a folder, labeled and separated, and the folder stored in the conservafile in a separate, labeled bin. After the S.I. is no longer needed, the folder shall be entered and stored in the MO comps files.

VCM/EB

SLOPE INDICATOR LOG

Location P.I.N. D.H. #

Date Start Date End

Station Elevation

Core Recovery %/5.0' Depth of Hole into Till

	Depth Below Ground	Stick Up Above Ground
6" Collar Pipe and Cap		
HW Casing to Till or Ledge		
S.I. Casing plus Tip		
Inside of S.I.		

Grout

Quantity Mixed Gals. % Mix Used

Water Gals. Cement Gals. Bentonite Gals.

Elevation of Grout Loss #1 Regrout Date

Elevation of Grout Loss #2 Regrout Date

Elevation of Grout Loss #3 Regrout Date

Comments on Hole Progression (Boulder Condition, Water Loss, Artesian Flow, etc.):

SLOPE INDICATOR SENSITIVITY

MODEL NOS. 50320 & 50325

CLAIMED SENSITIVITY : 1 IN 10,000

$\therefore \sin\theta = .0001$

$\theta = $ 20.6 SECONDS

= .0024 INCHES IN 2 FOOT READING LENGTH

SLOPE INDICATOR READING $= 2\sin\theta \times 10^4 =$ 2 UNITS

DIFFERENCE & CHANGE $= 4\sin\theta \times 10^4 =$ 4 UNITS

ACTUAL SENSITIVITY

$\theta = $ 3.4 MINUTES

= .024" IN 2 FOOT READING LENGTH

SLOPE INDICATOR READING = 20 UNITS

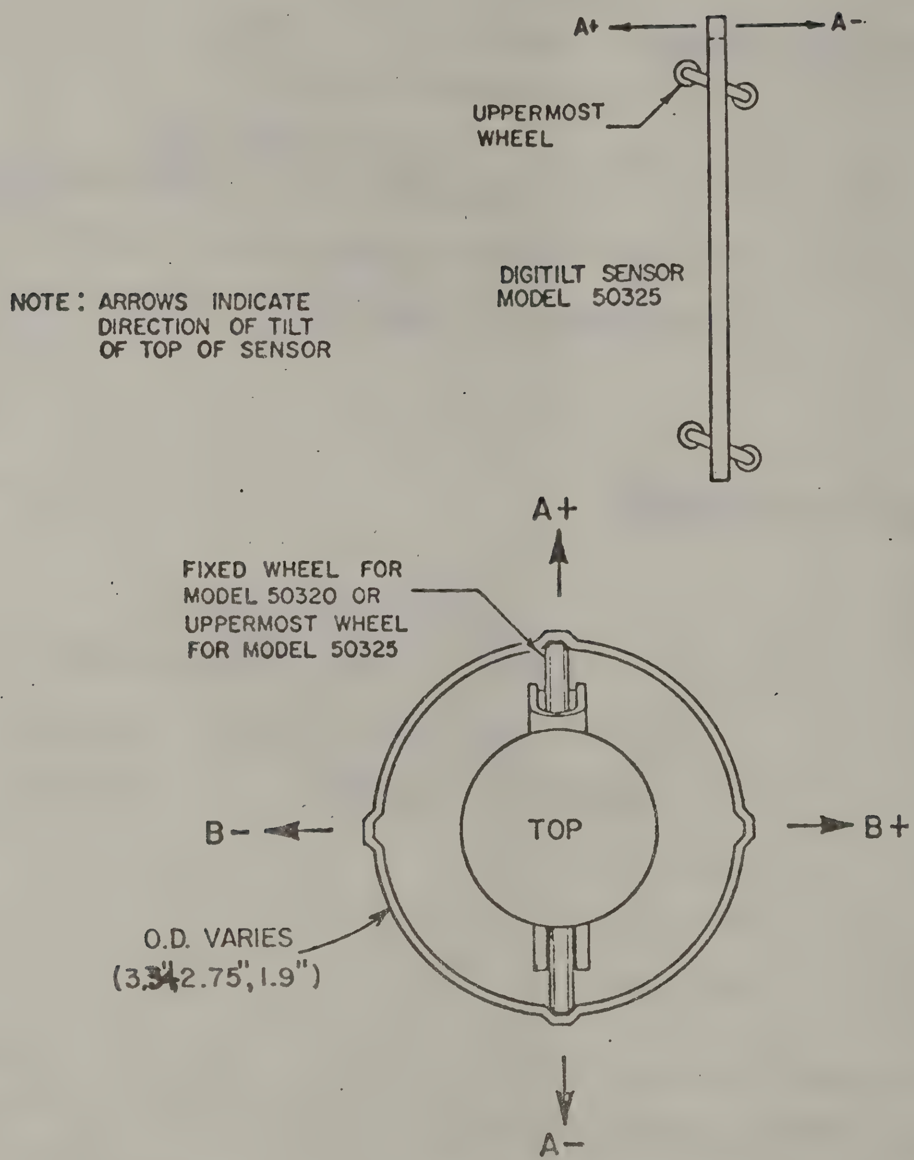
DIFFERENCE & CHANGE = 40 UNITS

NOTE : DEFLECTION = CHANGE $\times 6\times10^{-4} = 4\sin\theta \times 10^4 \times 6\times10^{-4}$

$= 24\sin\theta$ (INCHES) $= 2\sin\theta$ (FEET)

(FOR 2 FOOT READING LENGTH)

($\sin\theta$ FOR 1 FOOT READING LENGTH)

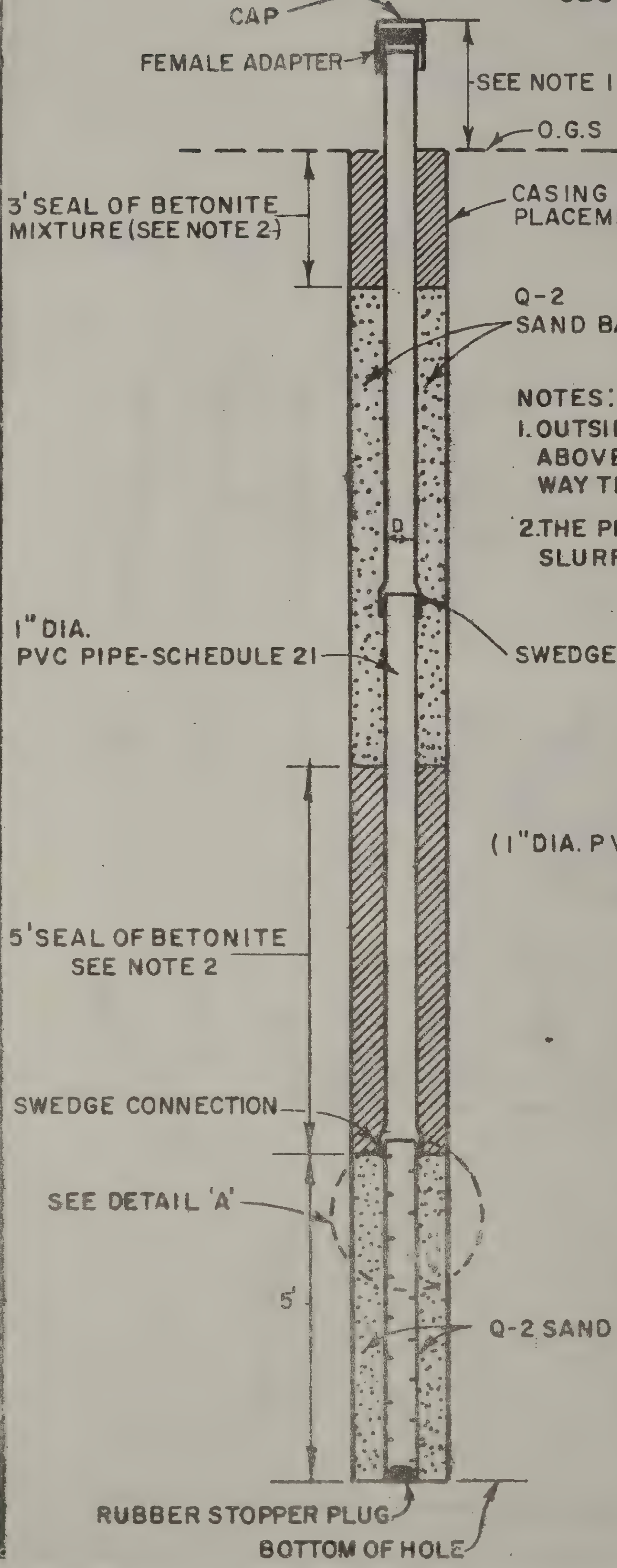

A+
A-
UPPERMOST
WHEEL
DIGITILT SENSOR
MODEL 50325
NOTE: ARROWS INDICATE
DIRECTION OF TILT
OF TOP OF SENSOR
A+
FIXED WHEEL FOR
MODEL 50320 OR
UPPERMOST WHEEL
FOR MODEL 50325
B-
TOP
B+
O.D. VARIES
(3.3",2.75",1.9")
A-

APPROXIMATE SLOPE INDICATOR CASING MOVEMENT LIMITATIONS FOR DIFFERENT PROBE MODELS

2 FOOT VERTICAL DISTANCE BETWEEN SUCCESSIVE READINGS (SINCO models)

MODEL NO.	CASING SIZE	HORIZONTAL MOVEMENT: DUE TO BENDING		HORIZONTAL MOVEMENT: DUE TO SHEAR
SINCO, 50320	3.34" O.D.	$3\frac{1}{4}$", 5"	DIFFERENT READINGS DUE TO CASING BENDING IN OPPOSITE DIRECTIONS	2"
	2.75" O.D.	$3\frac{1}{4}$"	DIFFERENT FIXED WHEELS AND REORIENTATION OF FLEXIBLE WHEELS USED FOR DIFFERENT CASING SIZES	$1\frac{1}{2}$"
SINCO 50325	3.34" O.D.	$5\frac{1}{4}$"		$1\frac{3}{4}$"
	2.75" O.D.	$3\frac{3}{4}$"		$1\frac{1}{4}$"
	1.90" O.D.	$1\frac{1}{2}$"		$\frac{1}{2}$"
POOR MAN'S	1" I.D.	4"	PROBE LENGTH = 3"	$\frac{1}{2}$"
	1" I.D.	2"	PROBE LENGTH = 6"	$\frac{1}{2}$"
	1" I.D.	1"	PROBE LENGTH = 12"	$\frac{1}{2}$"
	1" I.D.	$\frac{1}{2}$"	PROBE LENGTH = 24"	$\frac{1}{2}$"
	1" I.D.	$\frac{1}{4}$"	PROBE LENGTH = 36"	$\frac{1}{2}$"

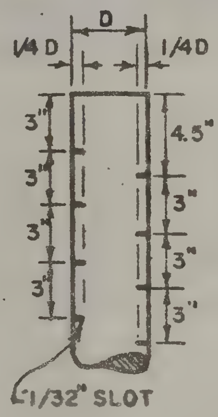
TYPICAL SECTIONS
INSTALLATION PROCEDURES FOR GROUND WATER OBSERVATION WELLS



NOTES:

1. OUTSIDE THE ROADWAY THE CAP SHALL BE 1 FT. ABOVE ORIGINAL GROUND. WHEN IN THE TRAVELED WAY THE CAP SHOULD BE EVEN WITH THE SURFACE.
2. THE PROCEDURE FOR MIXING AND PLACING THE SLURRY BETONITE SEAL WILL BE FORWARDED.

DETAIL 'A'
(1" DIA. PVC PIPE-SCHEDULE 21)



Prepared By: A. Minnith
Drawn By: J.J. MASI
Dwg. Reviewed By: A. Minnith
Checked By: R. L. Gemme

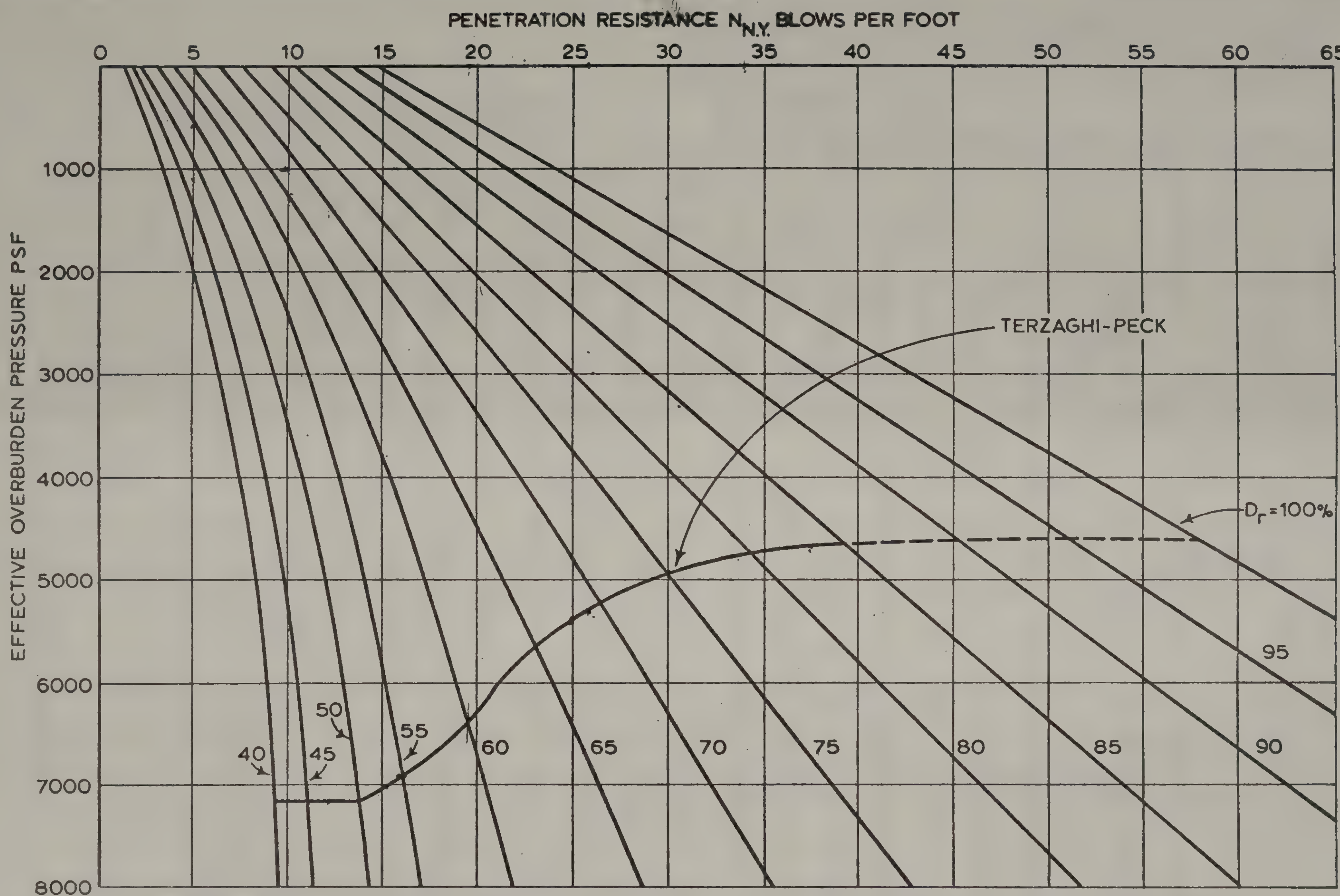
APPROVED Jan. 23 1973
L. H. Moore
DIRECTOR
SOIL MECHANICS BUREAU

REGION NO. 2
COUNTY: MONTGOMERY
DWG. NO. 2 SM 1735

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
DIVISION OF DESIGN AND CONSTRUCTION

P.I.N. 2006.07.101
FORT PLAIN - ROUTE 163

DRAWING 1 OF 1

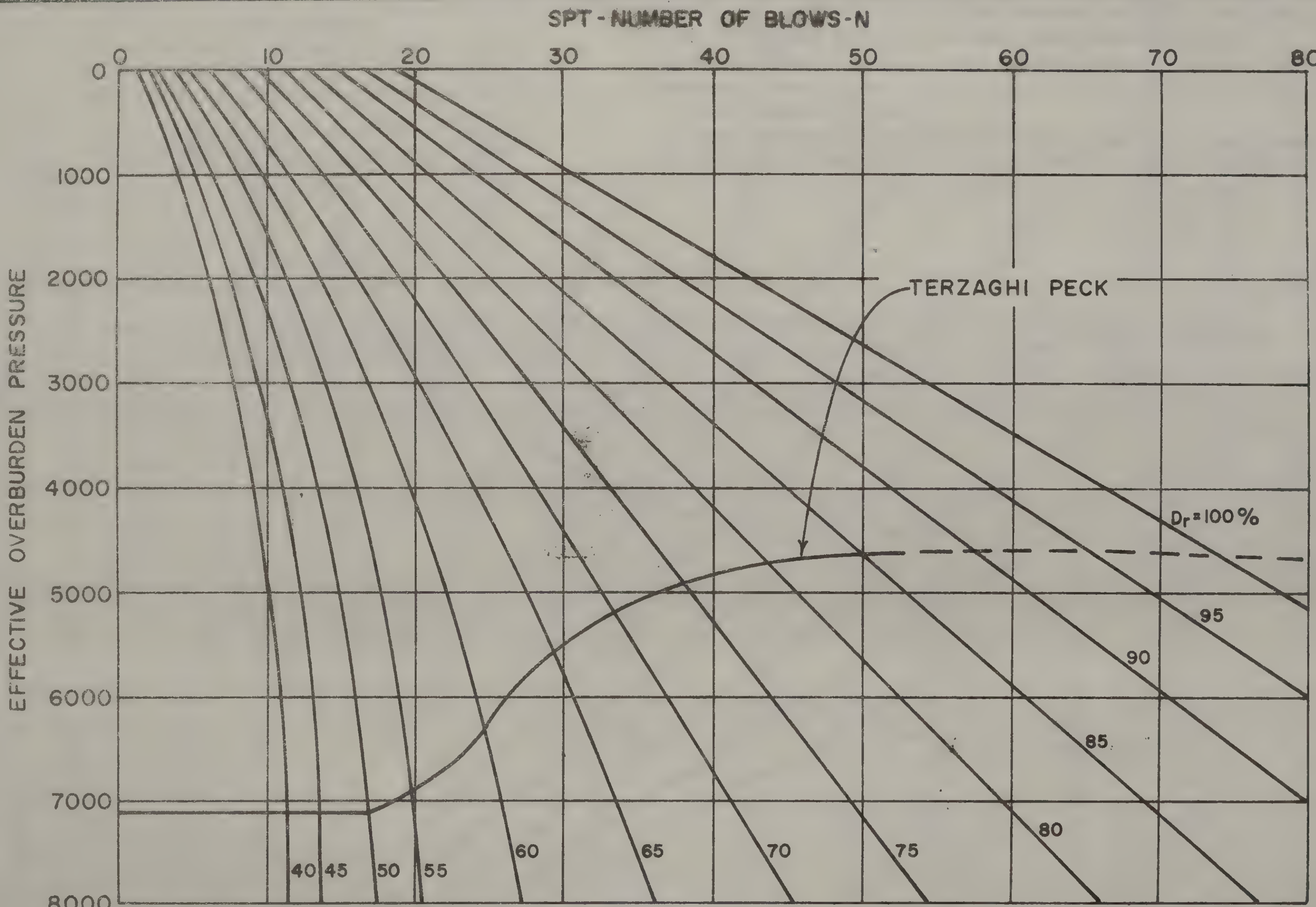


$N_{N.Y.}$ - Sampler blows for a 300 lb. hammer dropping 18" inches ($N_{std.} = N_{NY} \times 1.29$)

For dense fully saturated fine sands and silts with sample blows (N) greater than 15 modify "N" as follows. $N' = 15 + 1/2\,(N-15)$ (Approximate)

REF. I. ALPAN "Settlements of Foundations on Sand" Engineering and Public Works Review, November 1964. Also see Figure 4-1, Navdocks DM-7

THE INFLUENCE OF EFFECTIVE OVERBURDEN PRESSURE ON THE RELATIVE DENSITY OF COHESIONLESS MATERIAL.



For dense fully saturated fine sands and silts with sample blows (N) greater than 15 modify "N" as follows. $N' = 15 + 1/2(N-15)$

(Approximate)

REF. I. ALPAN "Settlements of Foundations on Sand" Engineering and Public Works Review, November 1964. Also see Figure 4-1, Navdocks DM-7

THE INFLUENCE OF EFFECTIVE OVERBURDEN PRESSURE ON THE RELATIVE DENSITY OF COHESIONLESS MATERIAL.

SECTION 5

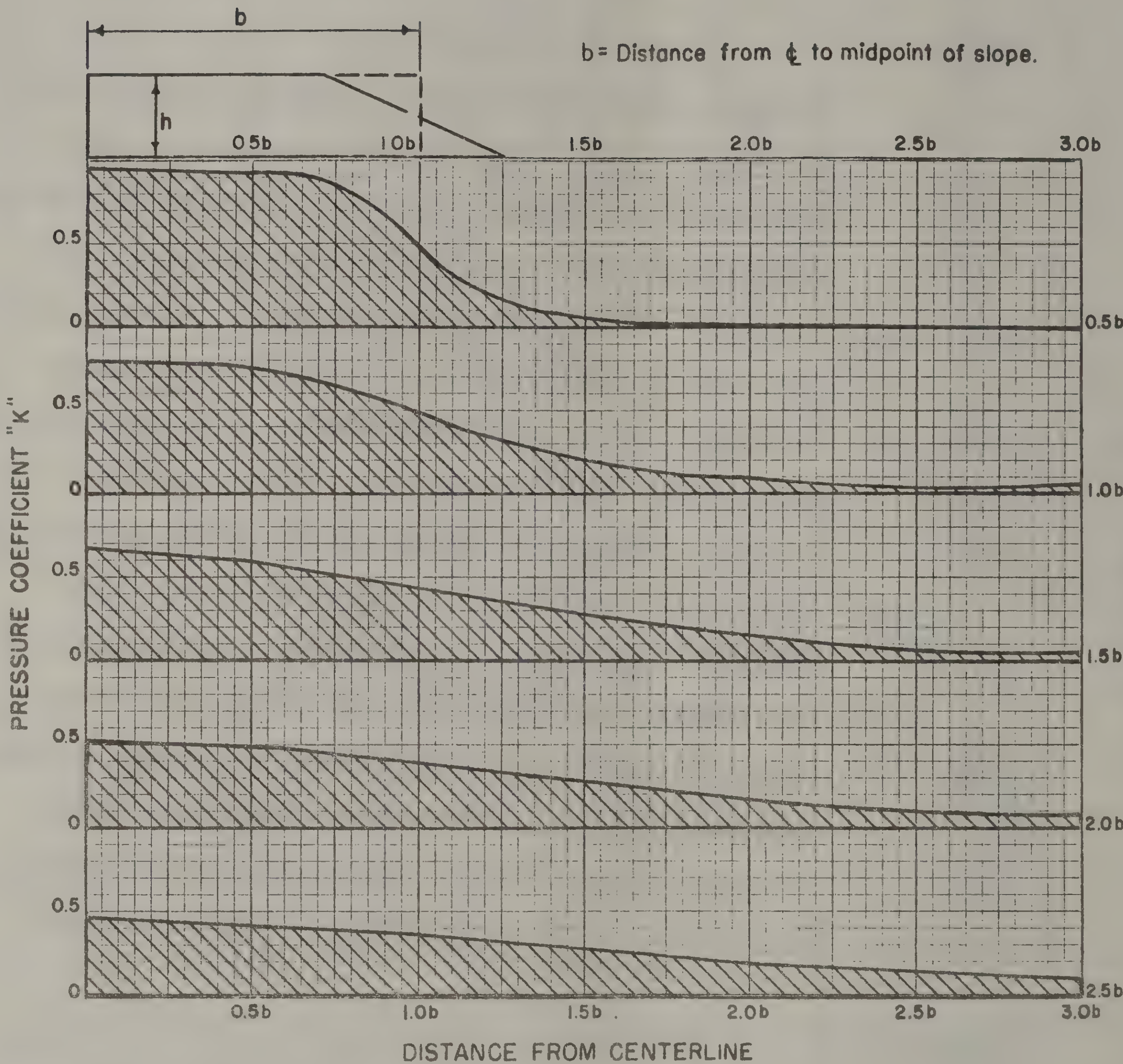
DISTRIBUTION OF STRESSES AND PRESSURES

PAGES

MANUALS

PRESSURE DISTRIBUTION CHART

CASE OF LONG STRIP–UNIFORM LOADING
VERTICAL PRESSURES UNDER TRANSVERSE SECTION



Note: From article in Journal of BOSTON SOCIETY OF CIVIL ENGINEERS July 1934 by Leo Jurgenson

PRESSURE COEFFICIENTS UNDER A LONG STRIP UNIFORM LOAD

5-2

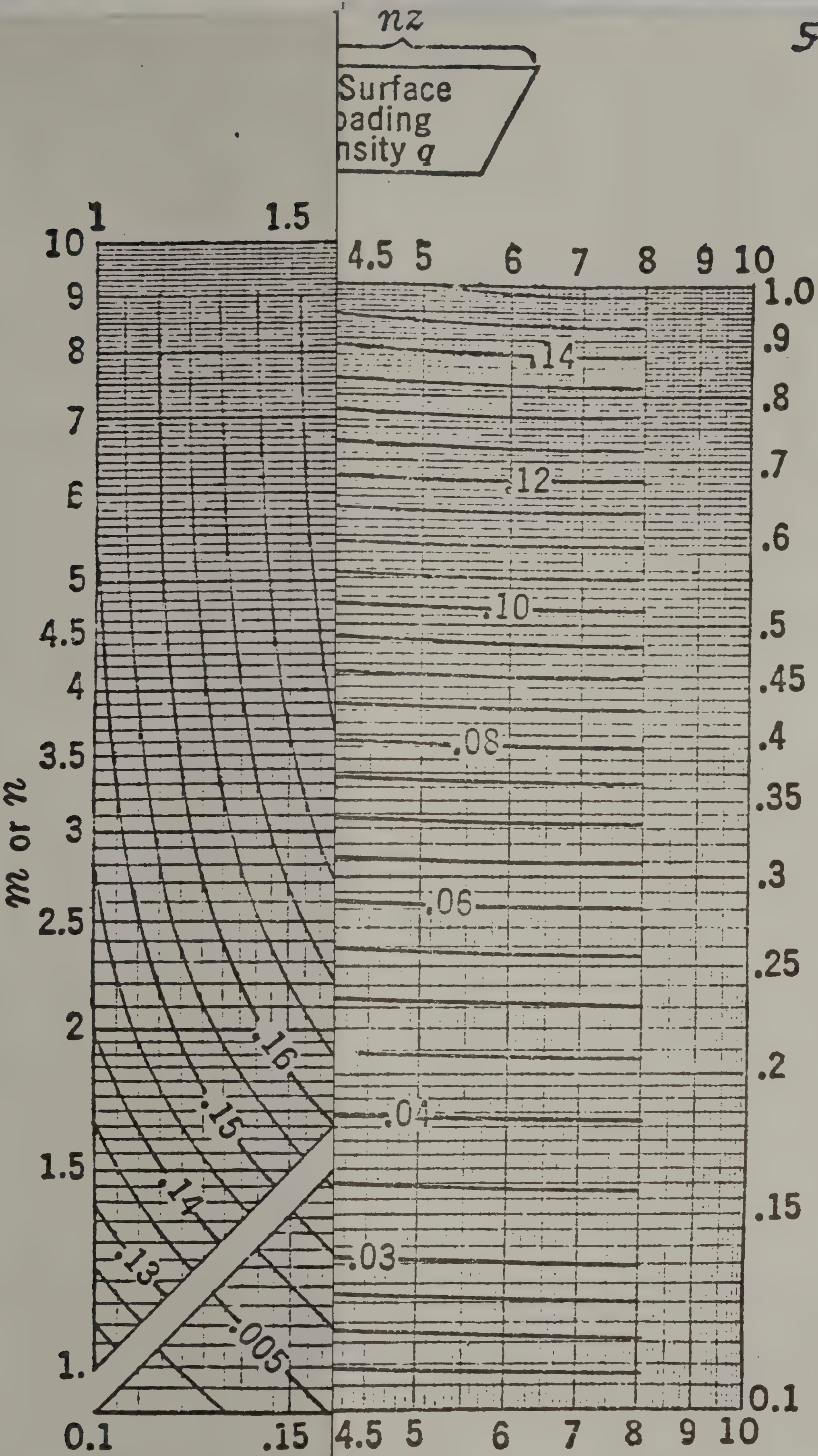
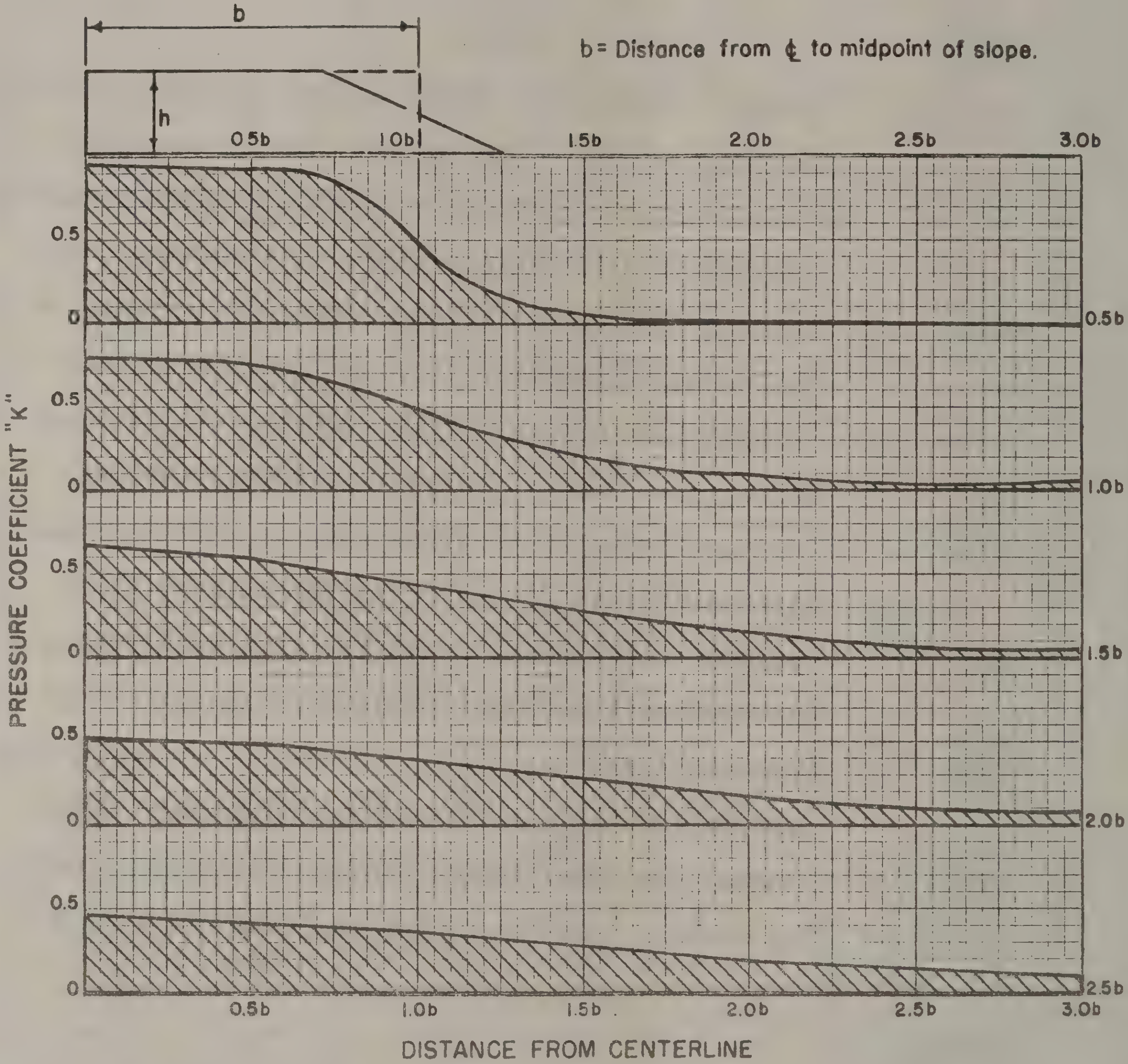


FIG. 11·5 Char surface areas, based on the q × $f_W(m, n)$.

PRESSURE DISTRIBUTION CHART

CASE OF LONG STRIP–UNIFORM LOADING
VERTICAL PRESSURES UNDER TRANSVERSE SECTION



Note: From article in Journal of BOSTON SOCIETY OF CIVIL ENGINEERS July 1934 by Leo Jurgenson

PRESSURE COEFFICIENTS UNDER A LONG STRIP UNIFORM LOAD

5-2

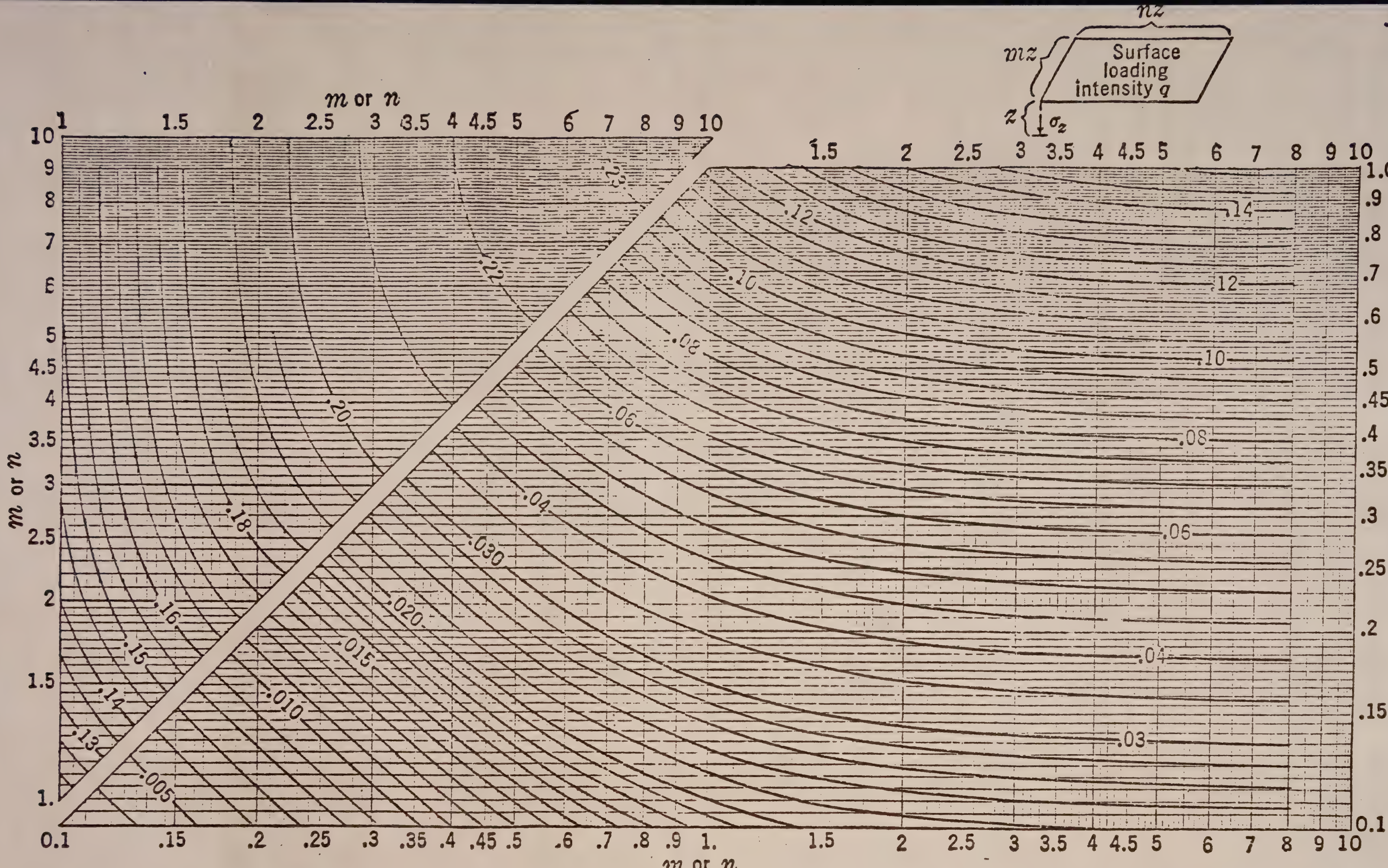


FIG. 11·5 Chart for use in determining vertical stresses below corners of loaded rectangular surface areas, based on the Westergaard (no lateral strain) elastic case. Chart gives $f_W(m, n)$; $\sigma_z = q \times f_W(m, n)$.

5-3

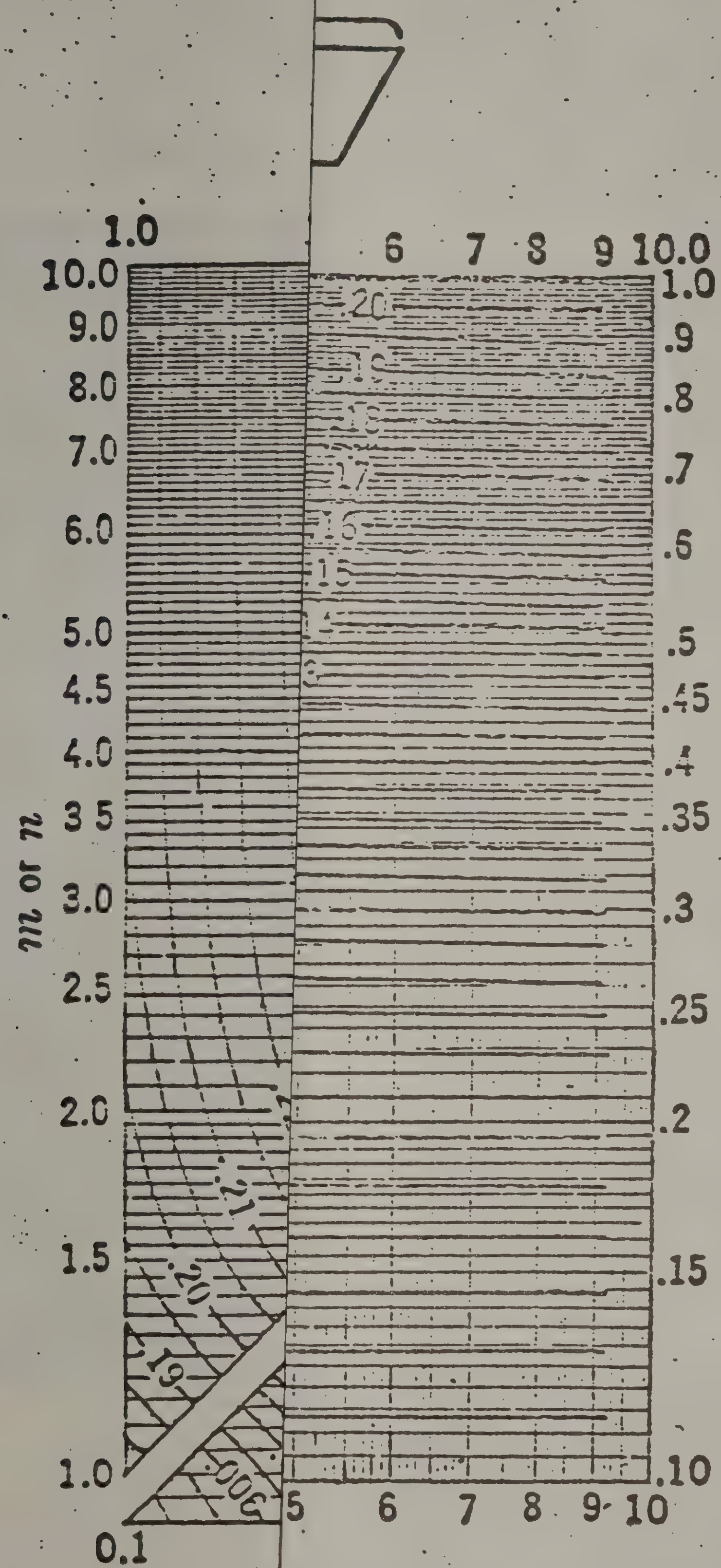


FIG. 11·3 C face areas on elastic,

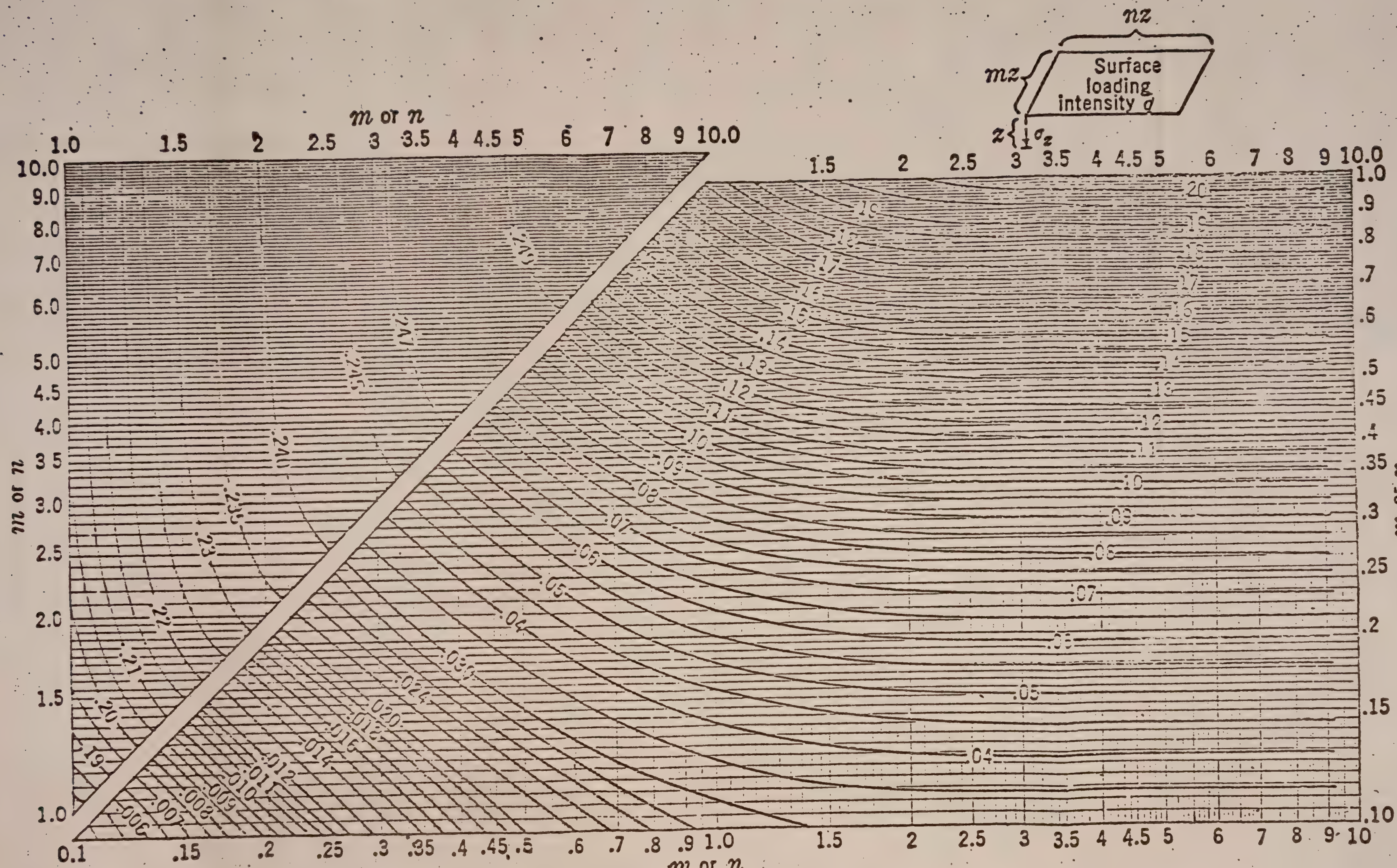
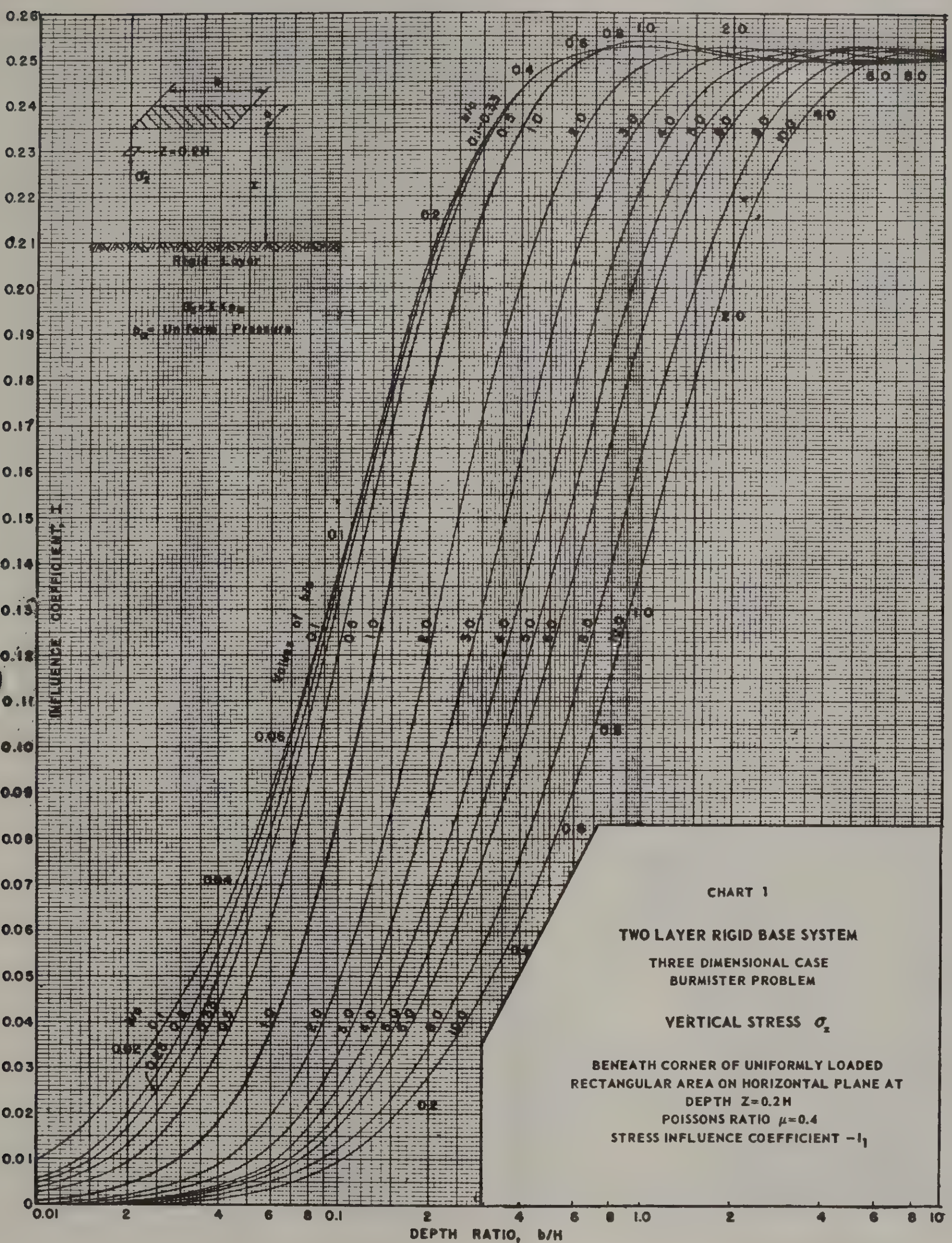


Fig. 11·3 Chart for use in determining vertical stresses below corners of loaded rectangular surface areas on elastic, isotropic material. Chart gives $f_B(m,n)$; $\sigma_z = q \times f_B(m,n)$.

S-4



INFLUENCE COEFFICIENT, I
DEPTH RATIO, b/H
Rigid Layer
$\sigma_z = I \times p_0$
p_0 = Uniform Pressure
Values of b/a
CHART 1
TWO LAYER RIGID BASE SYSTEM
THREE DIMENSIONAL CASE
BURMISTER PROBLEM
VERTICAL STRESS σ_z
BENEATH CORNER OF UNIFORMLY LOADED
RECTANGULAR AREA ON HORIZONTAL PLANE AT
DEPTH Z=0.2H
POISSONS RATIO μ=0.4
STRESS INFLUENCE COEFFICIENT $-I_1$

5-5

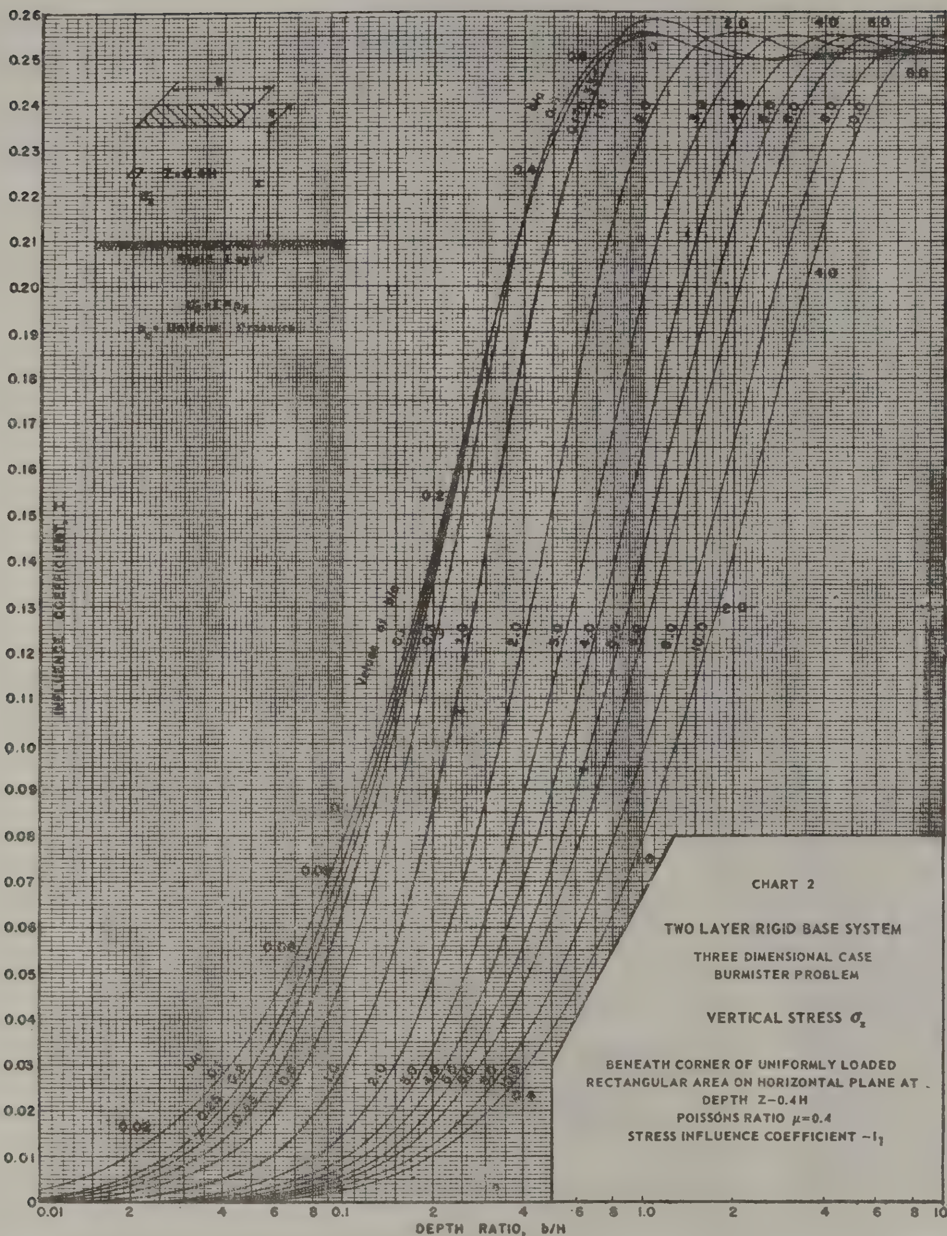


CHART 2
TWO LAYER RIGID BASE SYSTEM
THREE DIMENSIONAL CASE
BURMISTER PROBLEM
VERTICAL STRESS σ_z
BENEATH CORNER OF UNIFORMLY LOADED
RECTANGULAR AREA ON HORIZONTAL PLANE AT
DEPTH Z=0.4H
POISSONS RATIO μ=0.4
STRESS INFLUENCE COEFFICIENT ~I_1
INFLUENCE COEFFICIENT, I
DEPTH RATIO, b/H
Values of a/b

5-6

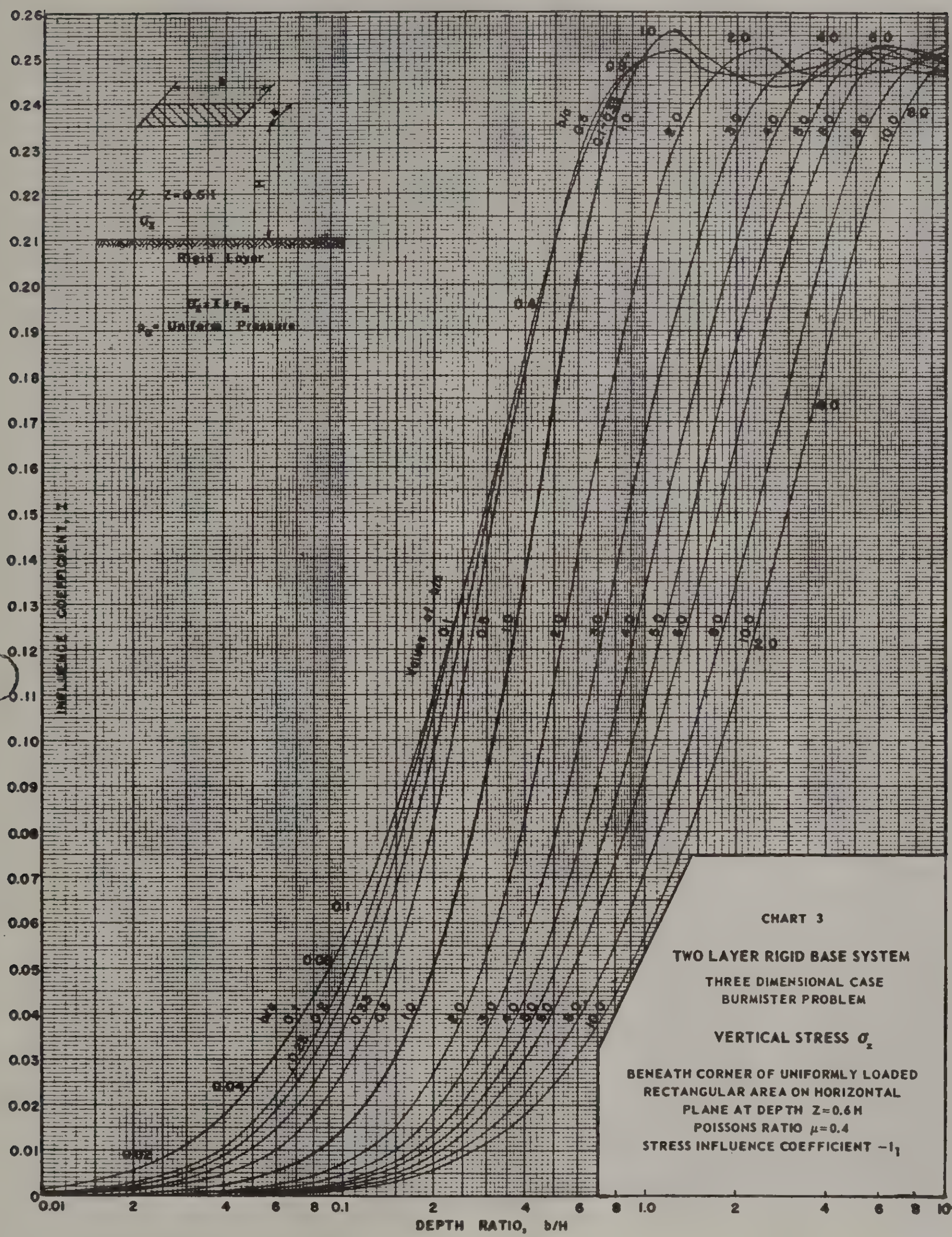


CHART 3
TWO LAYER RIGID BASE SYSTEM
THREE DIMENSIONAL CASE
BURMISTER PROBLEM
VERTICAL STRESS σ_z
BENEATH CORNER OF UNIFORMLY LOADED
RECTANGULAR AREA ON HORIZONTAL
PLANE AT DEPTH Z=0.6H
POISSONS RATIO μ=0.4
STRESS INFLUENCE COEFFICIENT $-I_1$
Rigid Layer
Z=0.6H
DEPTH RATIO, b/H
INFLUENCE COEFFICIENT, I
Values of a/b

5-7

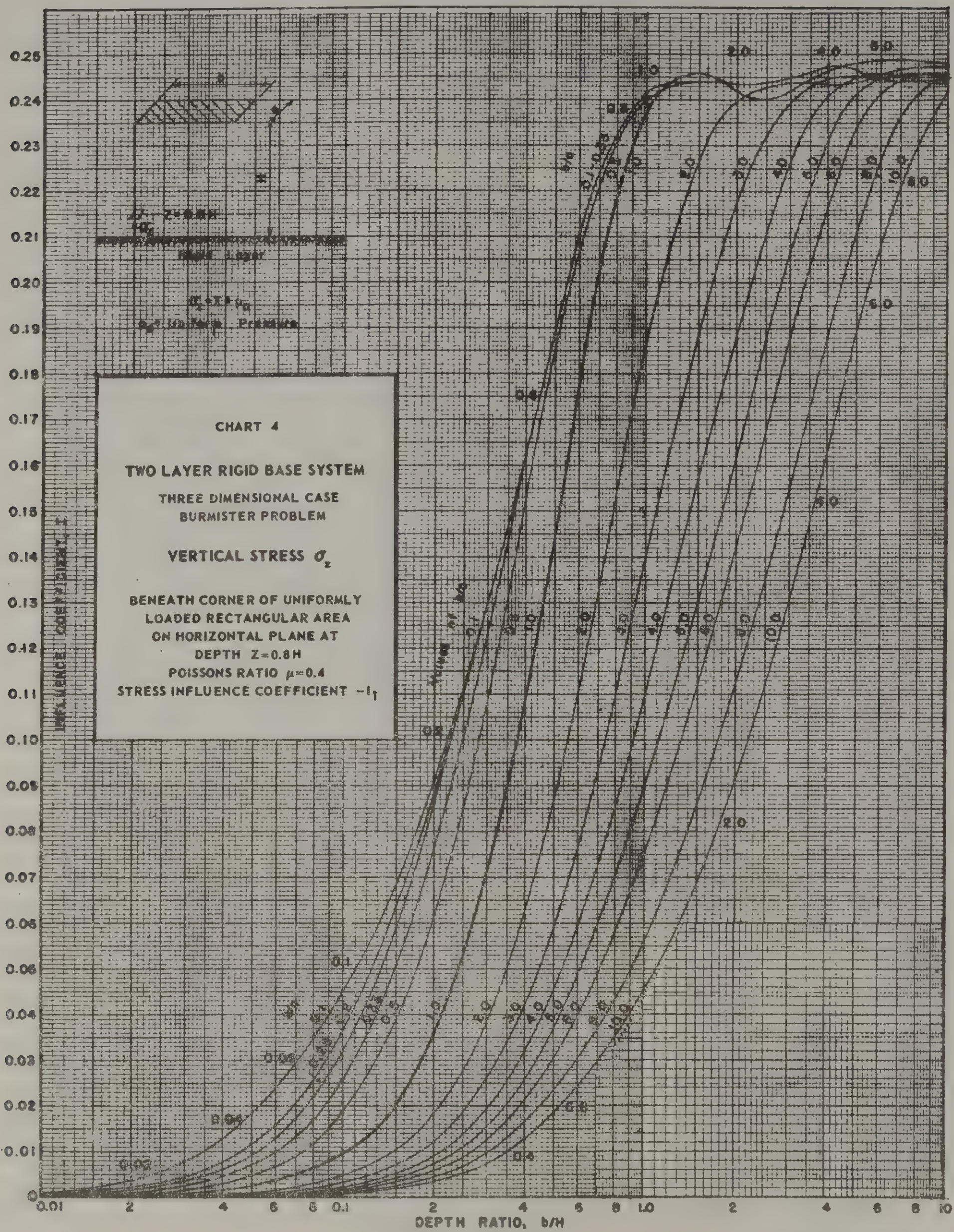


CHART 4
TWO LAYER RIGID BASE SYSTEM
THREE DIMENSIONAL CASE
BURMISTER PROBLEM
VERTICAL STRESS σ_z
BENEATH CORNER OF UNIFORMLY
LOADED RECTANGULAR AREA
ON HORIZONTAL PLANE AT
DEPTH Z=0.8H
POISSONS RATIO μ=0.4
STRESS INFLUENCE COEFFICIENT $-I_1$
INFLUENCE COEFFICIENT, I
DEPTH RATIO, b/H
Z=0.8H
Rigid Layer
Uniform Pressure
0.25
0.24
0.23
0.22
0.21
0.20
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0.01
2
4
6
8
0.1
2
4
6
8
1.0
2
4
6
8
10

5-8

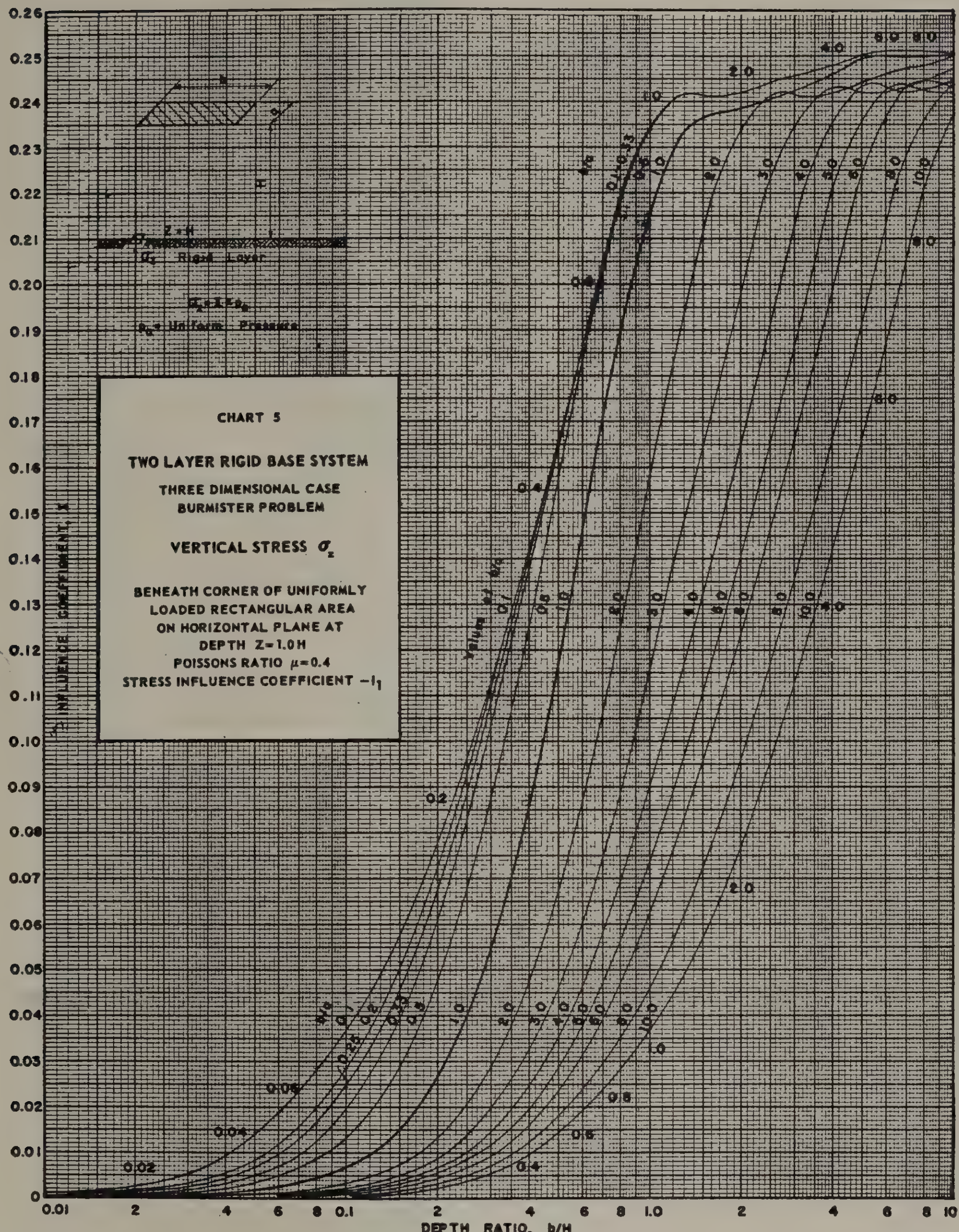
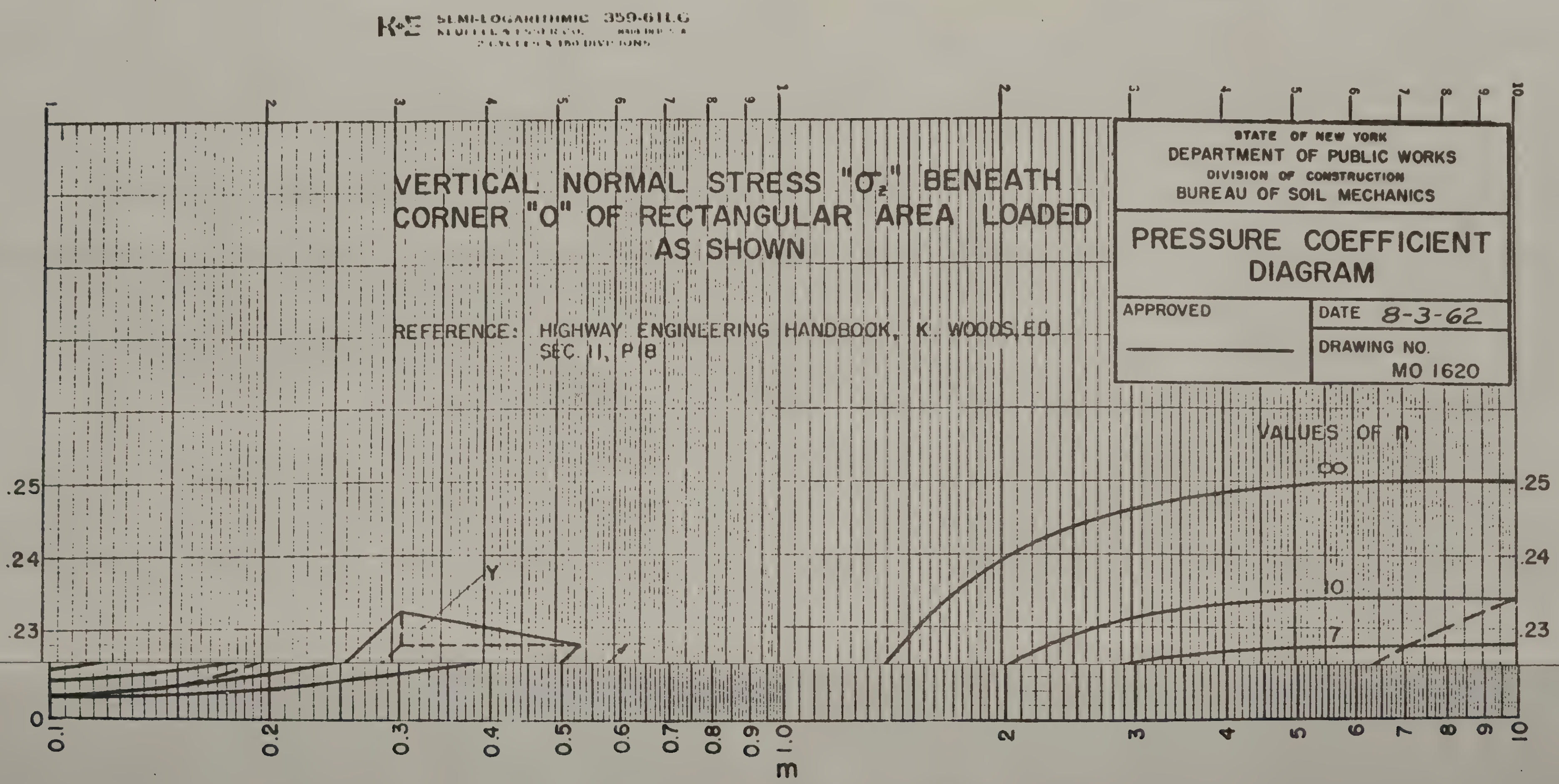
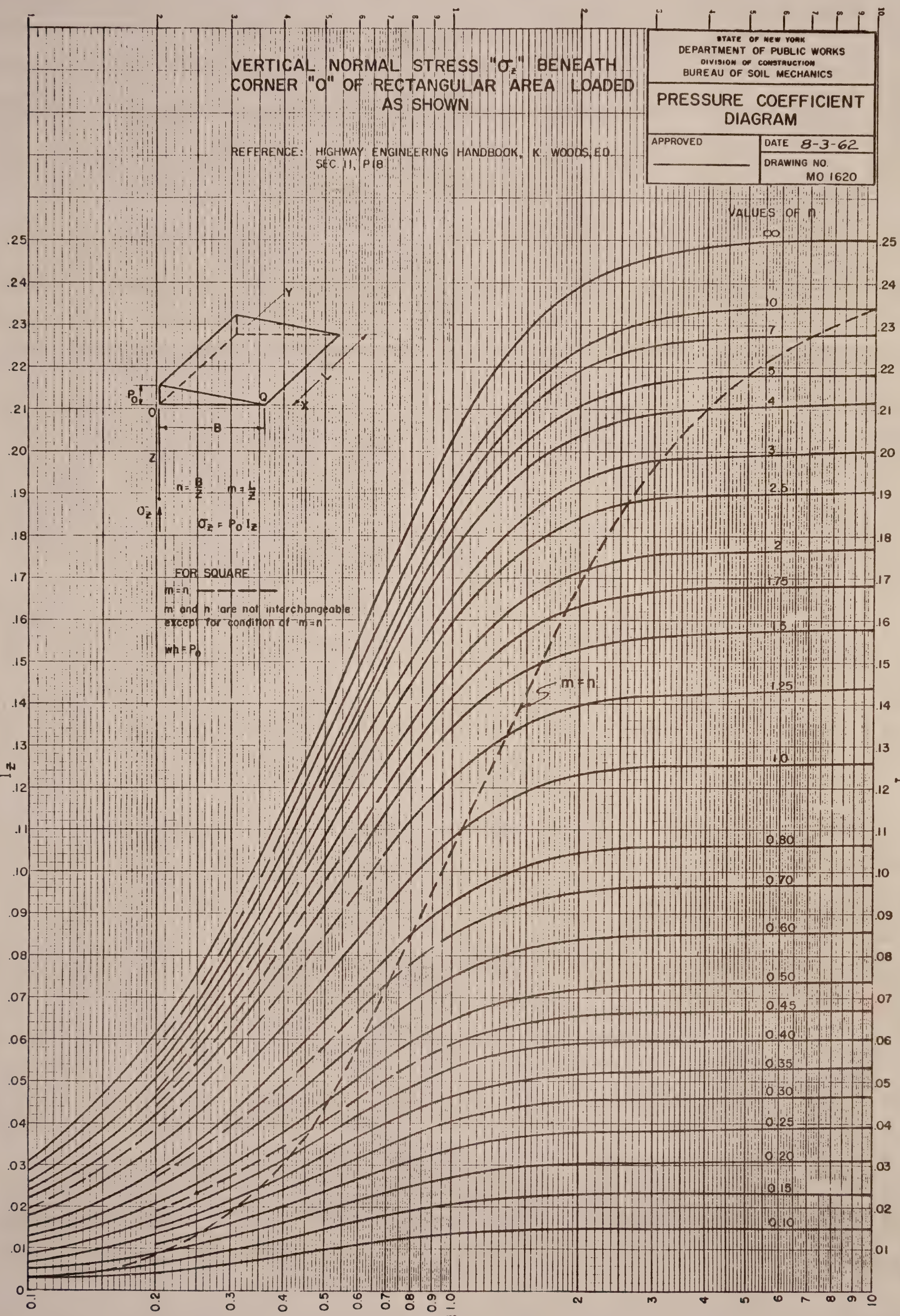
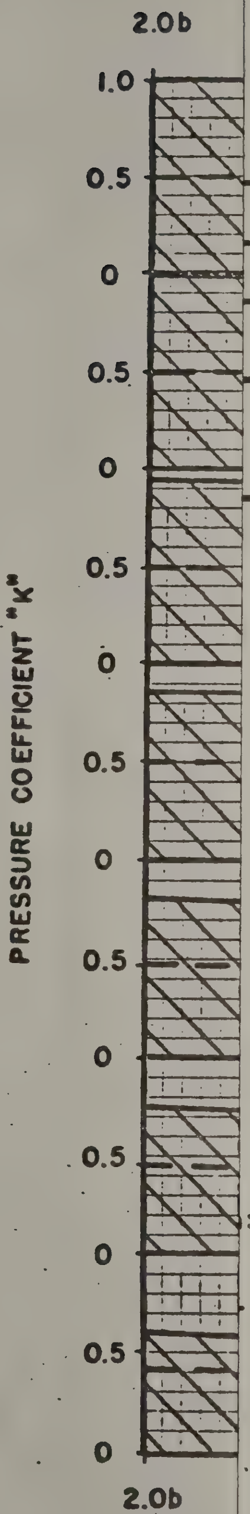
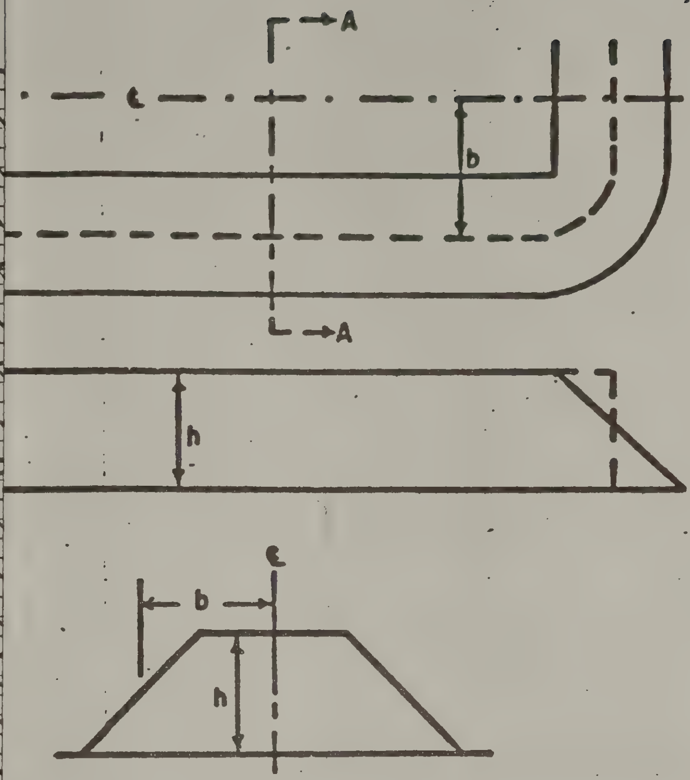


CHART 5
TWO LAYER RIGID BASE SYSTEM
THREE DIMENSIONAL CASE
BURMISTER PROBLEM
VERTICAL STRESS σ_z
BENEATH CORNER OF UNIFORMLY
LOADED RECTANGULAR AREA
ON HORIZONTAL PLANE AT
DEPTH Z=1.0H
POISSONS RATIO μ=0.4
STRESS INFLUENCE COEFFICIENT $-I_1$
INFLUENCE COEFFICIENT, K
DEPTH RATIO, b/H
Rigid Layer
Uniform Pressure
Values of a/b


K+E SEMI-LOGARITHMIC 359-61LG
VERTICAL NORMAL STRESS "σ_z" BENEATH CORNER "O" OF RECTANGULAR AREA LOADED AS SHOWN
REFERENCE: HIGHWAY ENGINEERING HANDBOOK, K. WOODS, ED. SEC. 11, P 18
STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS
PRESSURE COEFFICIENT DIAGRAM
APPROVED
DATE 8-3-62
DRAWING NO. MO 1620
VALUES OF n
∞
10
7
Y
.25
.24
.23
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2
3
4
5
6
7
8
9
10
m


K+E SEMI-LOGARITHMIC 359-61LG
VERTICAL NORMAL STRESS "σ_z" BENEATH CORNER "O" OF RECTANGULAR AREA LOADED AS SHOWN
REFERENCE: HIGHWAY ENGINEERING HANDBOOK, K. WOODS, ED. SEC. 11, P 18
STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS
PRESSURE COEFFICIENT DIAGRAM
APPROVED
DATE 8-3-62
DRAWING NO. MO 1620
VALUES OF n
$n = \frac{B}{Z}$ $m = \frac{L}{Z}$
$\sigma_z = P_0 I_z$
FOR SQUARE m=n
m and n are not interchangeable except for condition of m=n
$wh = P_0$
m=n
I_z
m





SECTION A–A

b = DISTANCE FROM ℄ TO MID-POINT OF SLOPE

FOR PRESSURE AT GREATER DEPTHS SEE LARGER CHART FOR VERTICAL PRESSURE BENEATH THE END OF A FILL DRAWING No. SM 1073

PRESSURE UNDER CENTER LINE = ———

PRESSURE UNDER EDGE OF RECTANGULAR FILL = ----

PRESSURE "p" AT ANY POINT = $K w_f h$

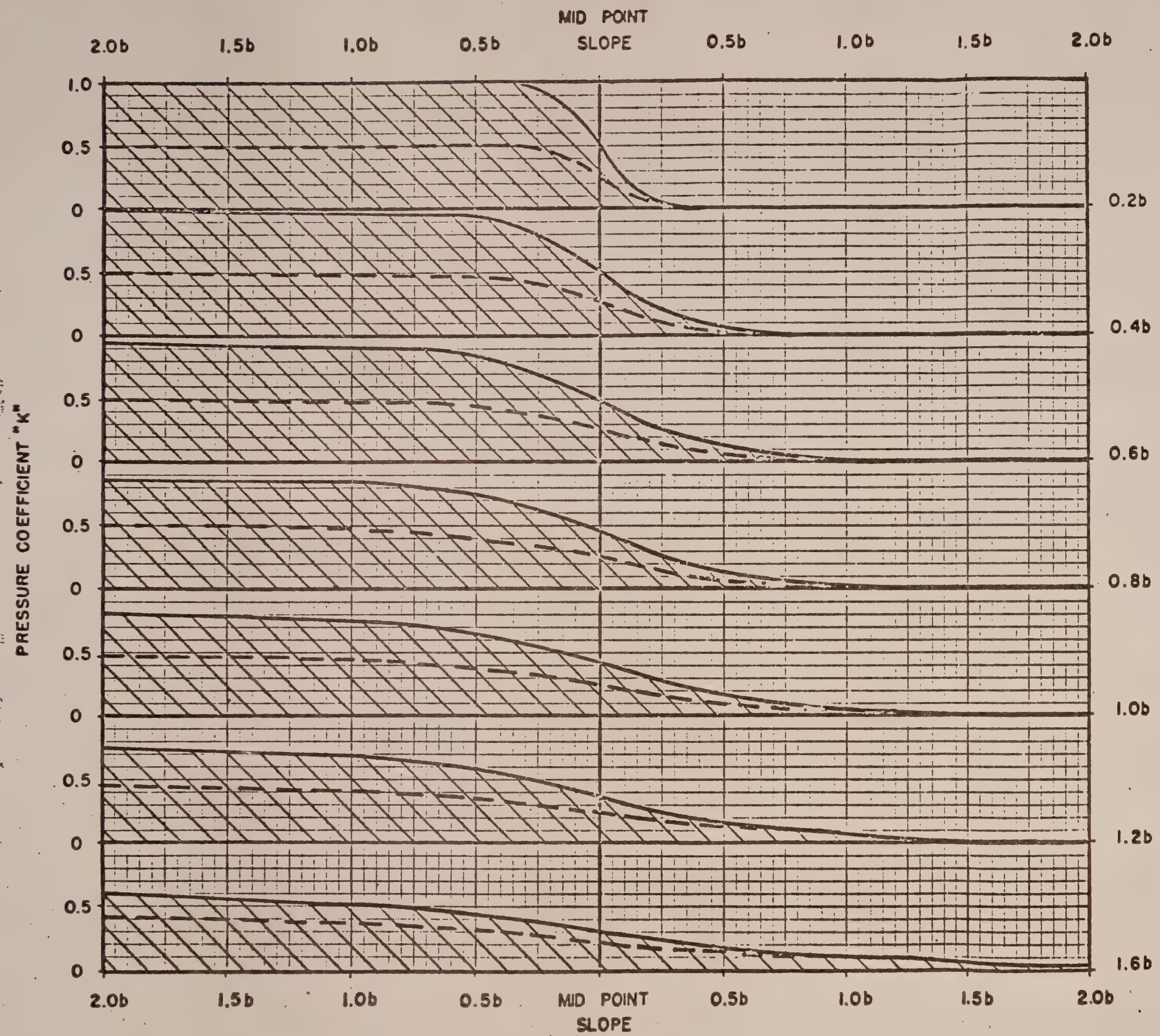
WHERE h = FILL HEIGHT

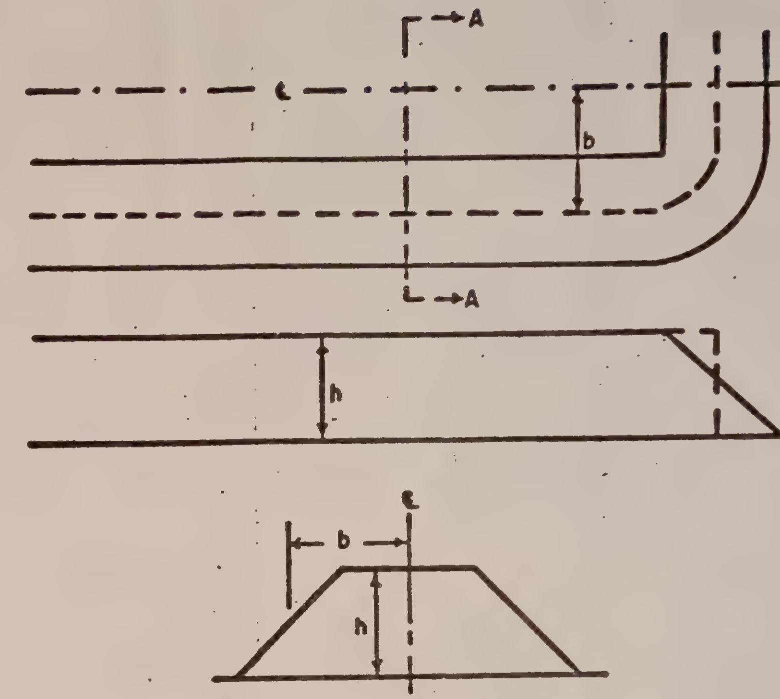
w_f = UNIT WEIGHT OF FILL MATERIAL

PRESSURE COEFFICIENTS BENEATH THE END OF A FILL

SN 1330

CASE OF LONG STRIP-UNIFORM LOADING
VERTICAL PRESSURES AT END OF FILL





SECTION A-A
b = DISTANCE FROM ℄ TO MID-POINT OF SLOPE

FOR PRESSURE AT GREATER DEPTHS SEE LARGER CHART FOR VERTICAL PRESSURE BENEATH THE END OF A FILL DRAWING No. SM 1073

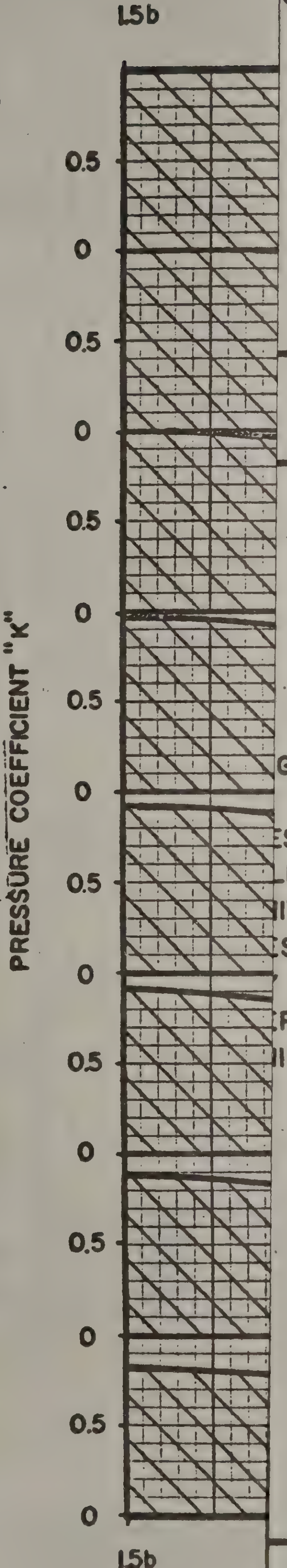
LEGEND: PRESSURE UNDER CENTER LINE = ———
PRESSURE UNDER EDGE OF RECTANGULAR FILL = ----
VERTICAL PRESSURE "p" AT ANY POINT = $Kw_f h$
WHERE h = FILL HEIGHT
w_f = UNIT WEIGHT OF FILL MATERIAL

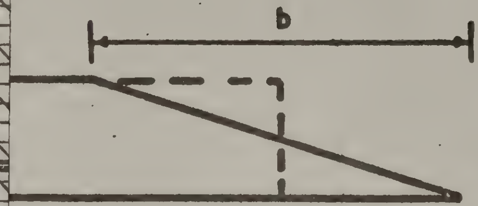
PRESSURE COEFFICIENTS
BENEATH THE END OF A FILL
SN 1330

CHART

LOADING

OF A TERRACE OR A CUT





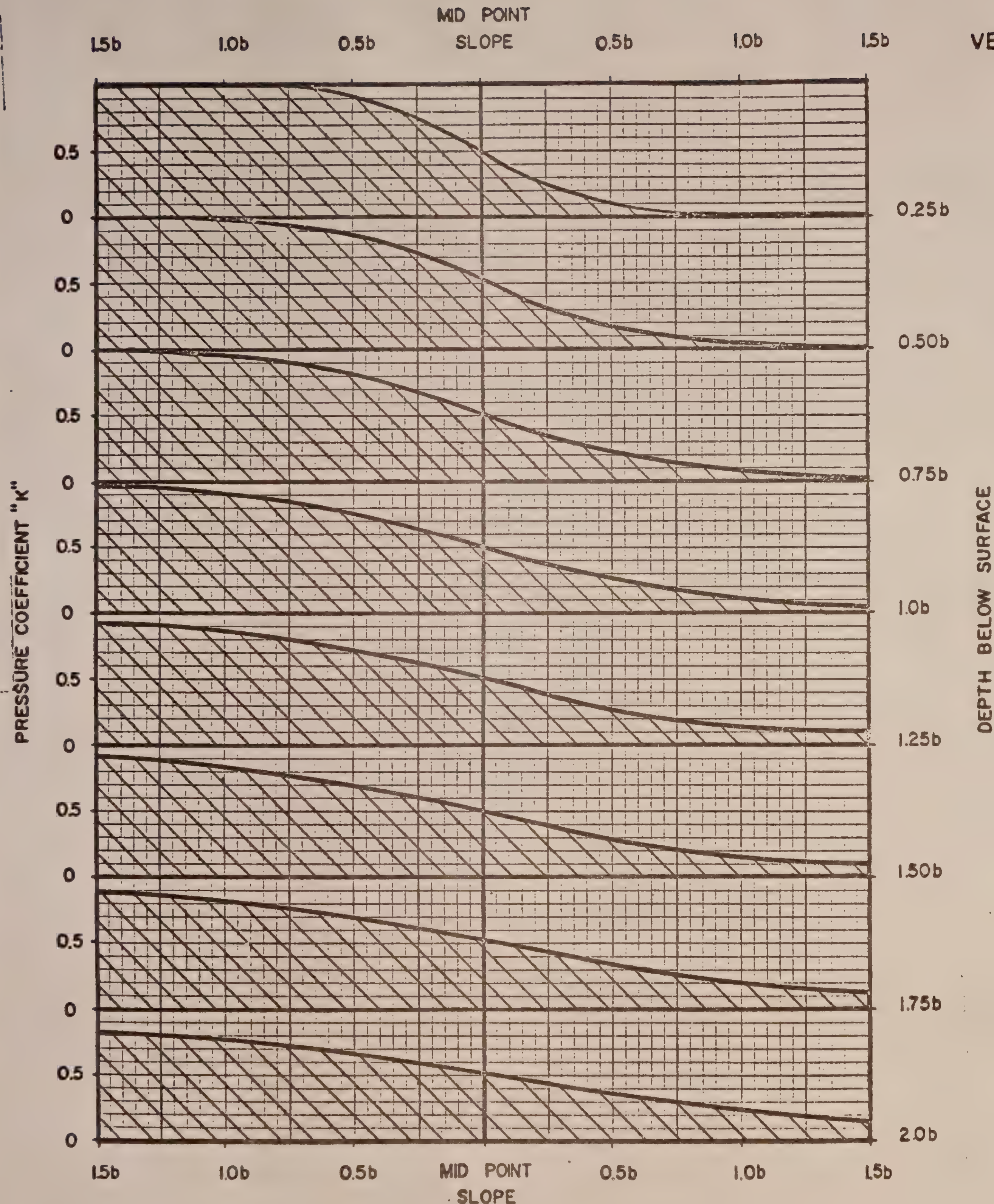
GREATER DEPTHS SEE LARGE CHART

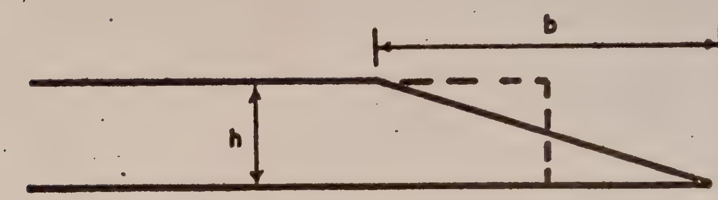
ESSURE "p" AT ANY POINT = $K w_f h$
LL HEIGHT
IIT WEIGHT OF FILL MATERIAL
ESSURE DECREASE CAUSED
Y BE COMPUTED IN SAME MANNER
EPTH OF CUT
IIT WEIGHT OF MATERIAL REMOVED

ESSURE COEFFICIENTS BENEATH THE
SLOPE OF A TERRACE OR A CUT

SM 1332

PRESSURE DISTRIBUTION CHART
CASE OF LONG STRIP-UNIFORM LOADING
VERTICAL PRESSURES BENEATH THE SLOPE OF A TERRACE OR A CUT



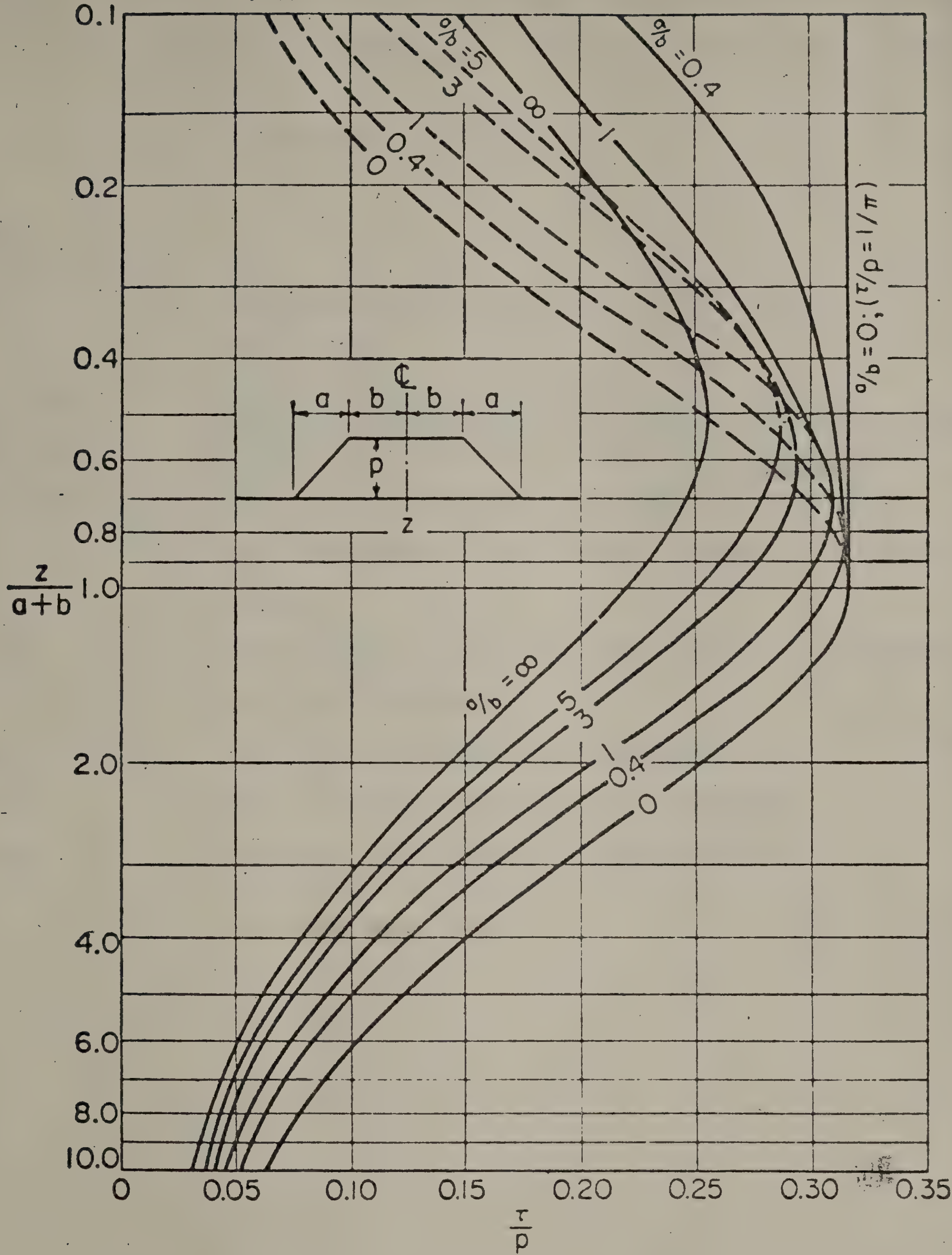


FOR PRESSURES AT GREATER DEPTHS SEE LARGE CHART DRAWING No. SM1074

NOTE: VERTICAL PRESSURE "p" AT ANY POINT = $K w_f h$
WHERE h = FILL HEIGHT
w_f = UNIT WEIGHT OF FILL MATERIAL
VERTICAL PRESSURE DECREASE CAUSED BY A CUT MAY BE COMPUTED IN SAME MANNER
WHERE h = DEPTH OF CUT
w_f = UNIT WEIGHT OF MATERIAL REMOVED

PRESSURE COEFFICIENTS BENEATH THE SLOPE OF A TERRACE OR A CUT

SM 1332



NOTE: DOTTED CURVES GIVE SHEARING STRESS ALONG CENTERLINE ONLY.

τ : MAXIMUM SHEARING STRESS

REF.: K.B. WOODS "HIGHWAY ENGINEERING HANDBOOK" FIG. 11-17, 1960

MAXIMUM SHEARING STRESS AT ANY DEPTH BENEATH EMBANKMENT LOAD

SECTION 6

SETTLEMENT ANALYSIS

$$U_z = 1 - \frac{U}{U_0} = 1 - \sum_{M=0}^{M=\infty} \frac{2}{M}\left(\sin \frac{Mz}{H}\right) \epsilon^{-M^2 T_V}$$

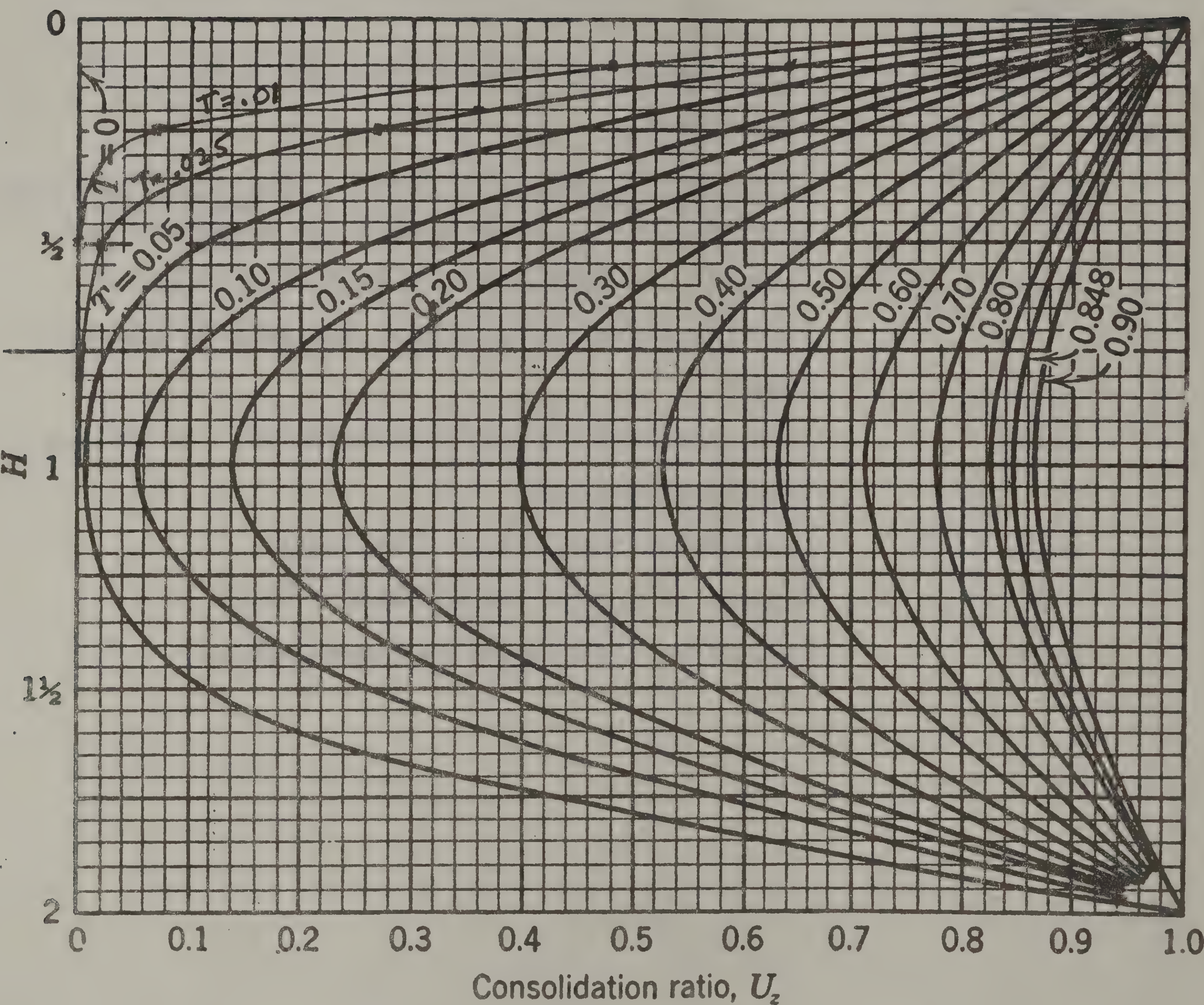
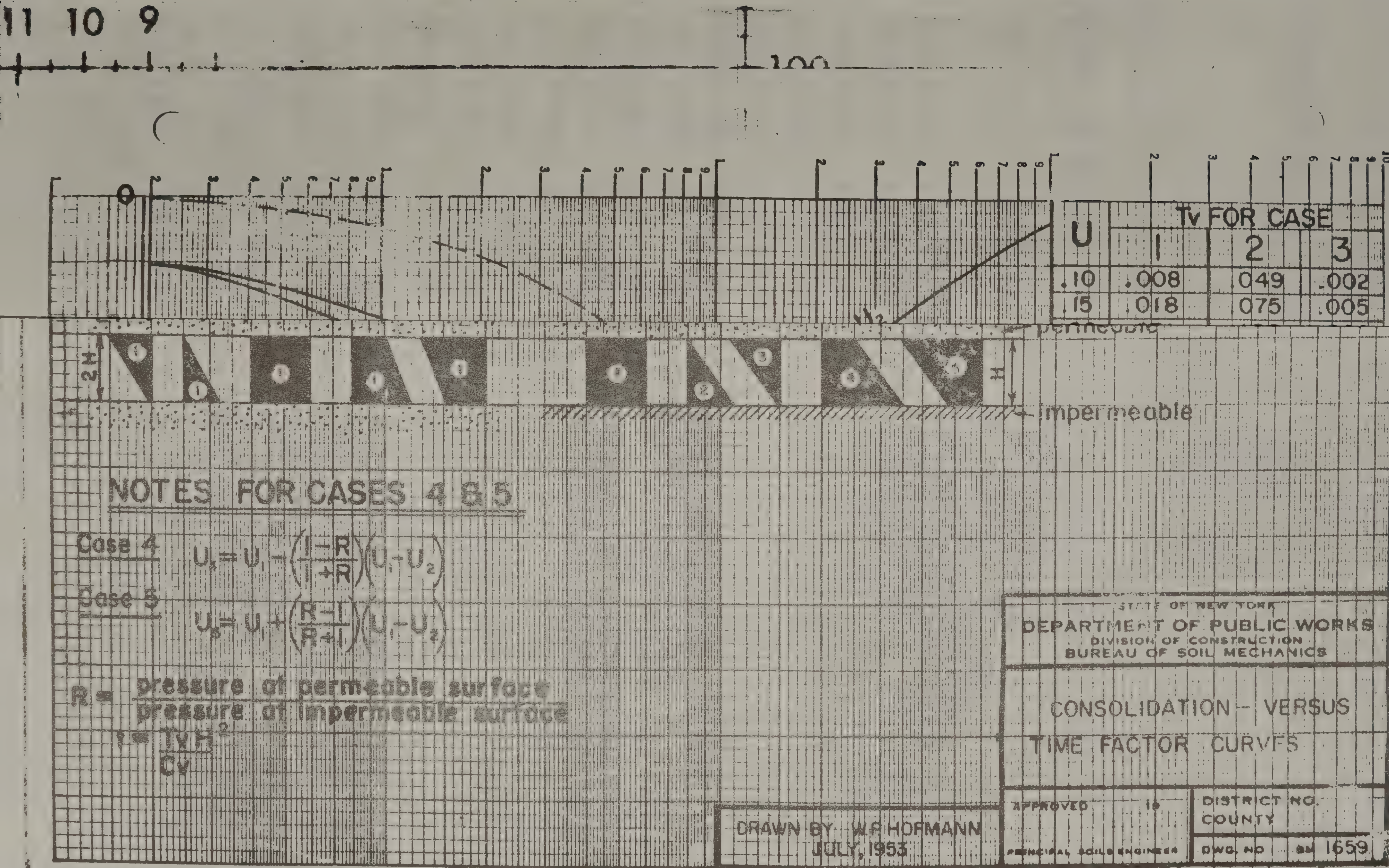


FIG. 10·9 Consolidation as a function of depth and time factor.

% U = % $\bar{P}$ AS RELATED TO PORE PRESSURE DISSIPATION AND STRENGTH GAIN

% U FOR SETTLEMENT COMPUTATIONS SHOULD BE DETERMINED FROM CHART ON PAGE 6-2B



11 10 9
100
U
Tv FOR CASE
1
2
3
.10 .008 .049 .002
.15 .018 .075 .005
permeable
impermeable
2H
H
NOTES FOR CASES 4 & 5
Case 4 $U_4 = U_1 - \left(\frac{1-R}{1+R}\right)(U_1 - U_2)$
Case 5 $U_5 = U_1 + \left(\frac{R-1}{R+1}\right)(U_1 - U_2)$
$R = \frac{\text{pressure of permeable surface}}{\text{pressure of impermeable surface}}$
$t = \frac{TvH^2}{Cv}$
STATE OF NEW YORK
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DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS
CONSOLIDATION - VERSUS
TIME FACTOR CURVES
APPROVED 19
PRINCIPAL SOILS ENGINEER
DISTRICT NO.
COUNTY
DWG. NO. SM 1659
DRAWN BY W.R HOFMANN
JULY, 1953

$$U_z = 1 - \frac{U}{U_o} = 1 - \sum_{M=0}^{M=\infty} \frac{2}{M}\left(\sin \frac{Mz}{H}\right) \epsilon^{-M^2 T_v}$$

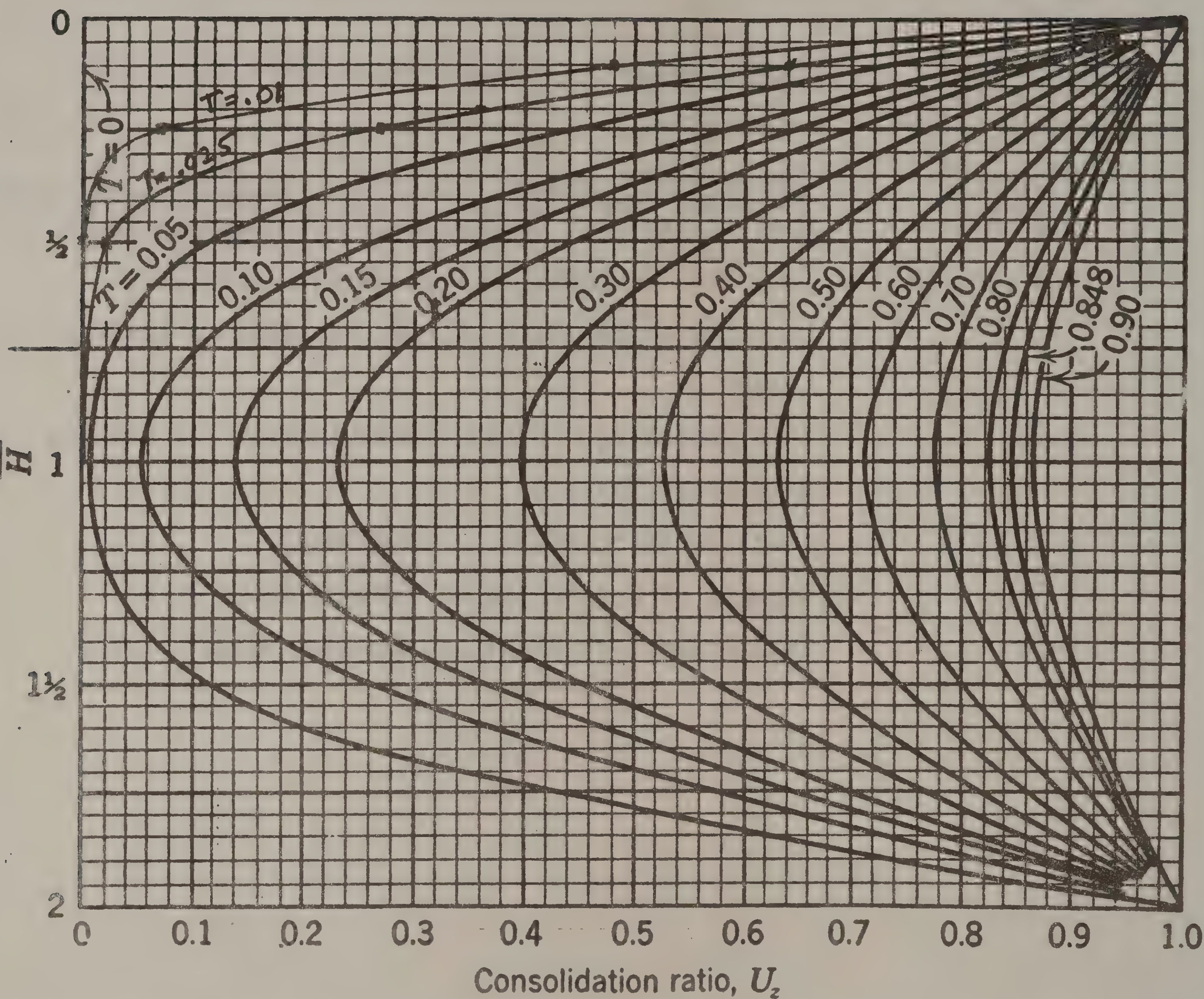
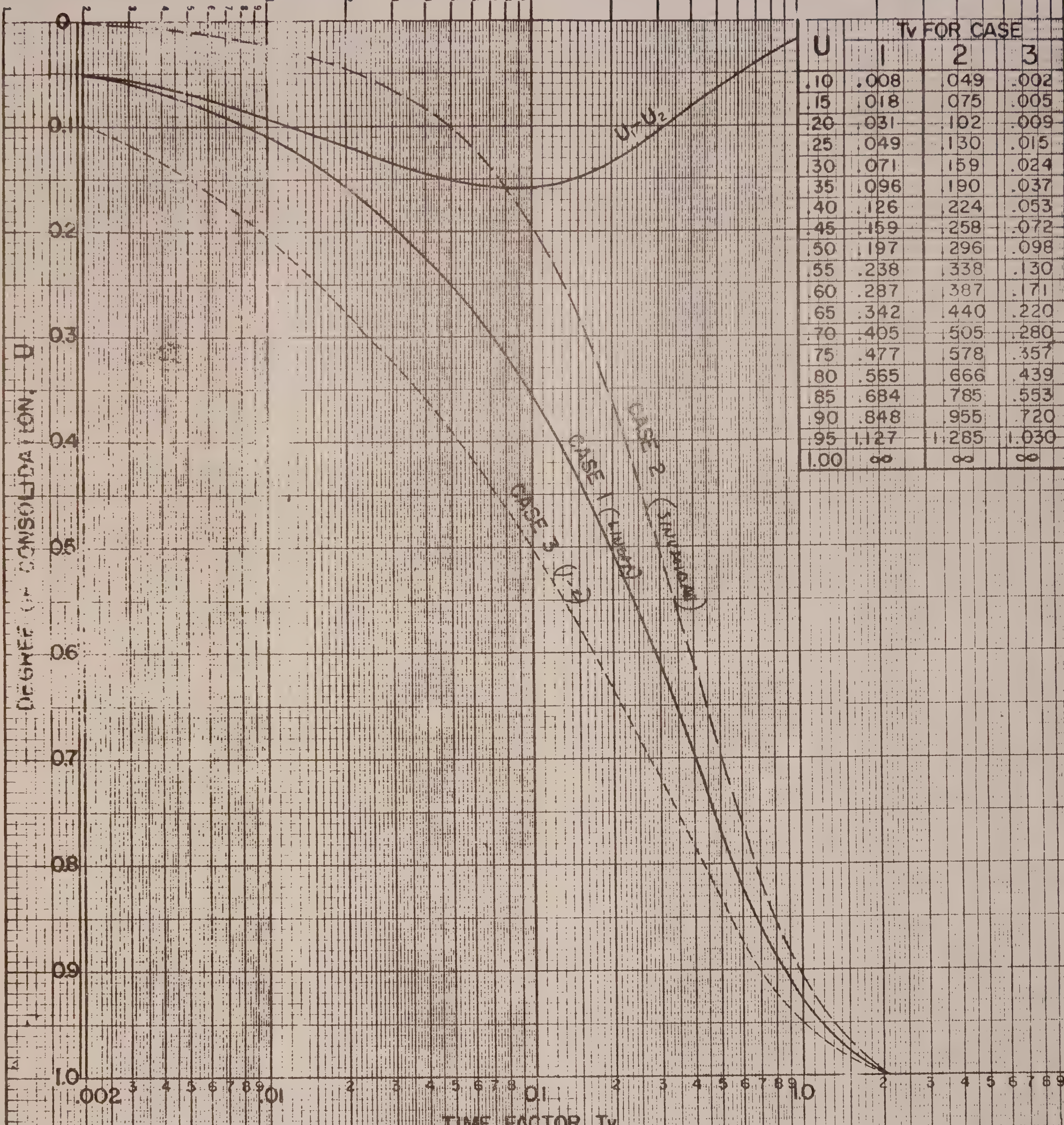


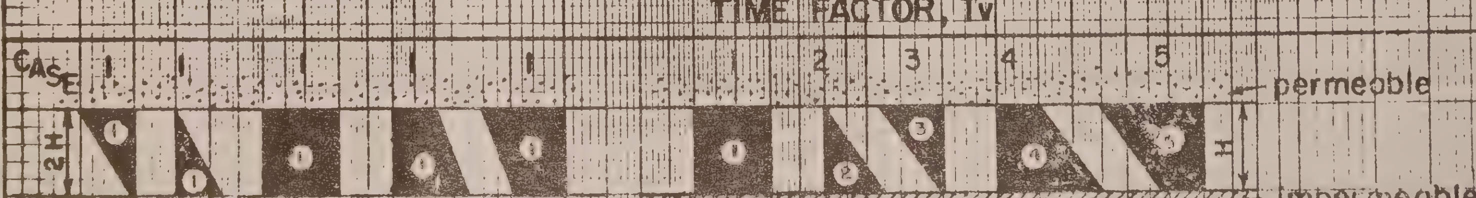
FIG. 10·9 Consolidation as a function of depth and time factor.

% U = % $\bar{P}$ AS RELATED TO PORE PRESSURE DISSIPATION AND STRENGTH GAIN

% U FOR SETTLEMENT COMPUTATIONS SHOULD BE DETERMINED FROM CHART ON PAGE 6-2B



U	Tv FOR CASE 1	Tv FOR CASE 2	Tv FOR CASE 3
.10	.008	.049	.002
.15	.018	.075	.005
.20	.031	.102	.009
.25	.049	.130	.015
.30	.071	.159	.024
.35	.096	.190	.037
.40	.126	.224	.053
.45	.159	.258	.072
.50	.197	.296	.098
.55	.238	.338	.130
.60	.287	.387	.171
.65	.342	.440	.220
.70	.405	.505	.280
.75	.477	.578	.357
.80	.565	.666	.439
.85	.684	.785	.553
.90	.848	.955	.720
.95	1.127	1.285	1.030
1.00	∞	∞	∞



NOTES FOR CASES 4 & 5

Case 4 $\quad U_4 = U_1 - \left(\frac{1-R}{1+R}\right)(U_1 - U_2)$

Case 5 $\quad U_5 = U_1 + \left(\frac{R-1}{R+1}\right)(U_1 - U_2)$

$R = \dfrac{\text{pressure of permeable surface}}{\text{pressure of impermeable surface}}$

$t = \dfrac{T_v H^2}{C_v}$

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CONSOLIDATION - VERSUS TIME FACTOR CURVES

DRAWN BY W.P. HOFMANN
JULY, 1953

APPROVED	19	DISTRICT NO.
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		DWG. NO. SM 1659

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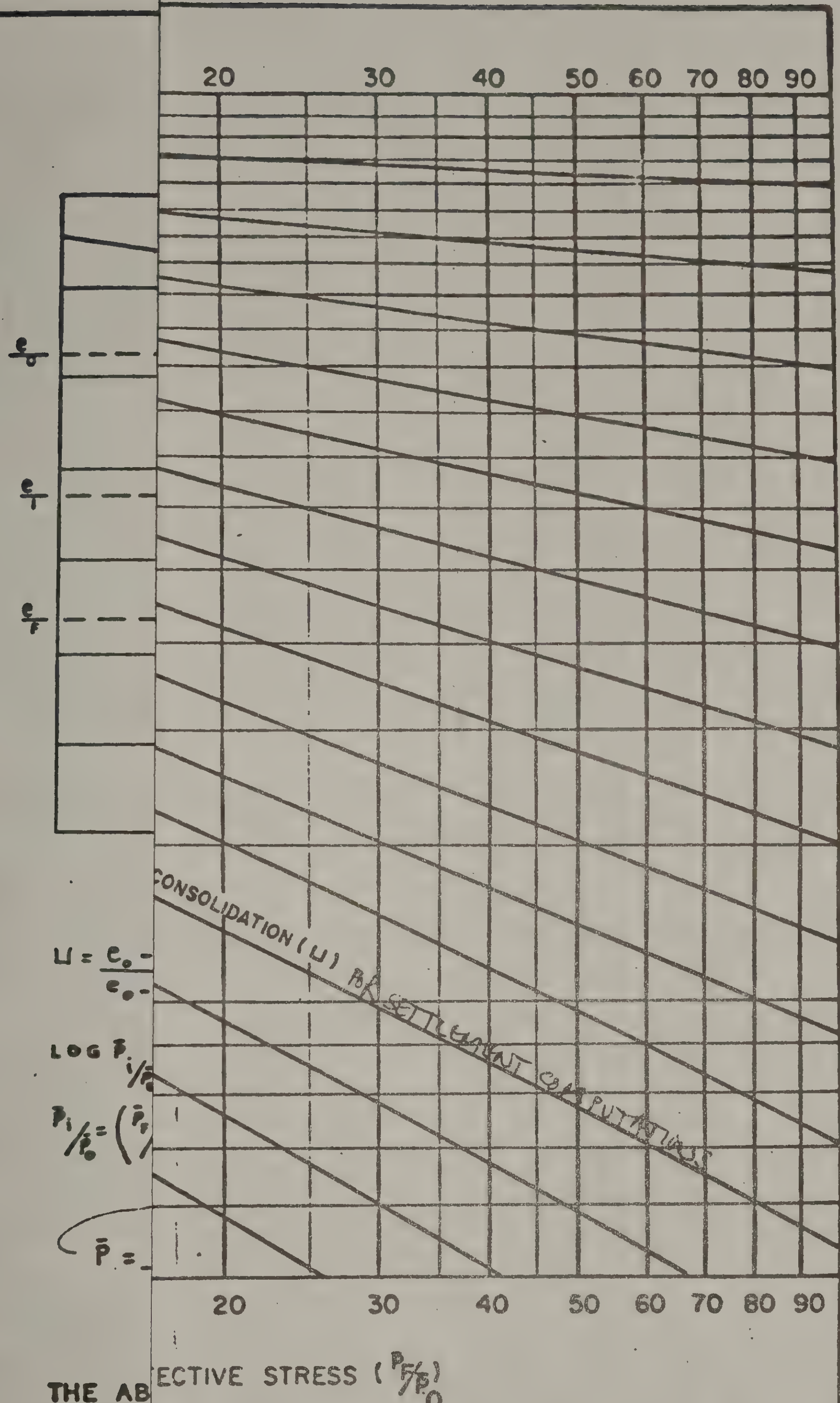
*****
PRINT U769
*****
FILE U769 /BØTØ (17, 17)
CONTAINS 21 RECORDS

CASE #1

%U	TV	%U	TV	%U	TV	%U	TV	%U	TV
0.01	0.000	0.21	0.035	0.41	0.132	0.61	0.297	0.81	0.588
0.02	0.000	0.22	0.038	0.42	0.139	0.62	0.307	0.82	0.610
0.03	0.001	0.23	0.042	0.43	0.145	0.63	0.318	0.83	0.633
0.04	0.001	0.24	0.045	0.44	0.152	0.64	0.329	0.84	0.658
0.05	0.002	0.25	0.049	0.45	0.159	0.65	0.340	0.85	0.684
0.06	0.003	0.26	0.053	0.46	0.166	0.66	0.352	0.86	0.712 30
0.07	0.004	0.27	0.057	0.47	0.173	0.67	0.364	0.87	0.742
0.08	0.005	0.28	0.062	0.48	0.181	0.68	0.377	0.88	0.774
0.09	0.006	0.29	0.066	0.49	0.189	0.69	0.390	0.89	0.809
0.10	0.008	0.30	0.071	0.50	0.196	0.70	0.403	0.90	0.848
0.11	0.010	0.31	0.075	0.51	0.204	0.71	0.417	0.91	0.891
0.12	0.011	0.32	0.080	0.52	0.212	0.72	0.431	0.92	0.939
0.13	0.013	0.33	0.086	0.53	0.221	0.73	0.446	0.93	0.993
0.14	0.015	0.34	0.091	0.54	0.229	0.74	0.461	0.94	1.055
0.15	0.018	0.35	0.096	0.55	0.238	0.75	0.477	0.95	1.129
0.16	0.020	0.36	0.102	0.56	0.246	0.76	0.493	0.96	1.219
0.17	0.023	0.37	0.108	0.57	0.255	0.77	0.511	0.97	1.336
0.18	0.025	0.38	0.113	0.58	0.264	0.78	0.529	0.98	1.500
0.19	0.028	0.39	0.119	0.59	0.273	0.79	0.547	0.00	0.000
0.20	0.031	0.40	0.126	0.60	0.287	0.80	0.565		

EOF REACHED

END CANDE



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STATE OF NEW YORK
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BUREAU OF SOIL MECHANICS

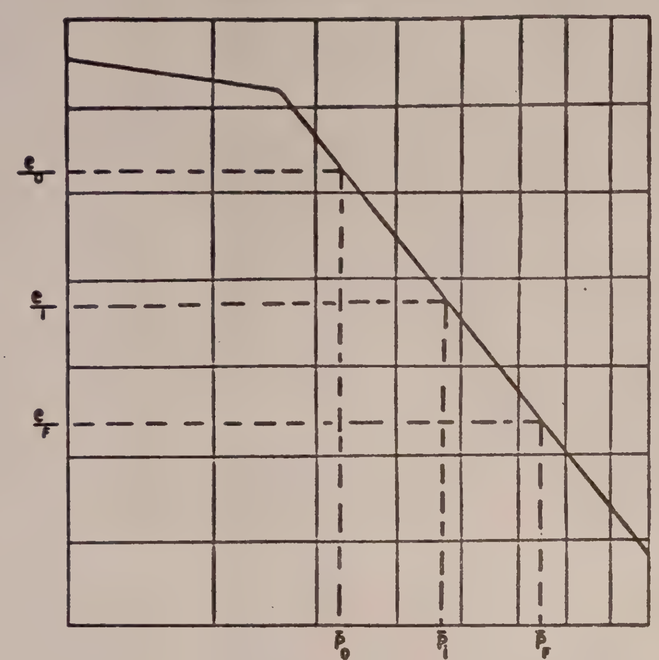
RELATIONSHIP BETWEEN PERCENT CONSOLIDATION AND PERCENT EFFECTIVE STRESS

APPROVED 4/18 1961
Wm. P. Hofmann
DIRECTOR

DISTRICT NO.
COUNTY MO

DWG NO SM 1691

TYPICAL e VS. LOG p PLOT



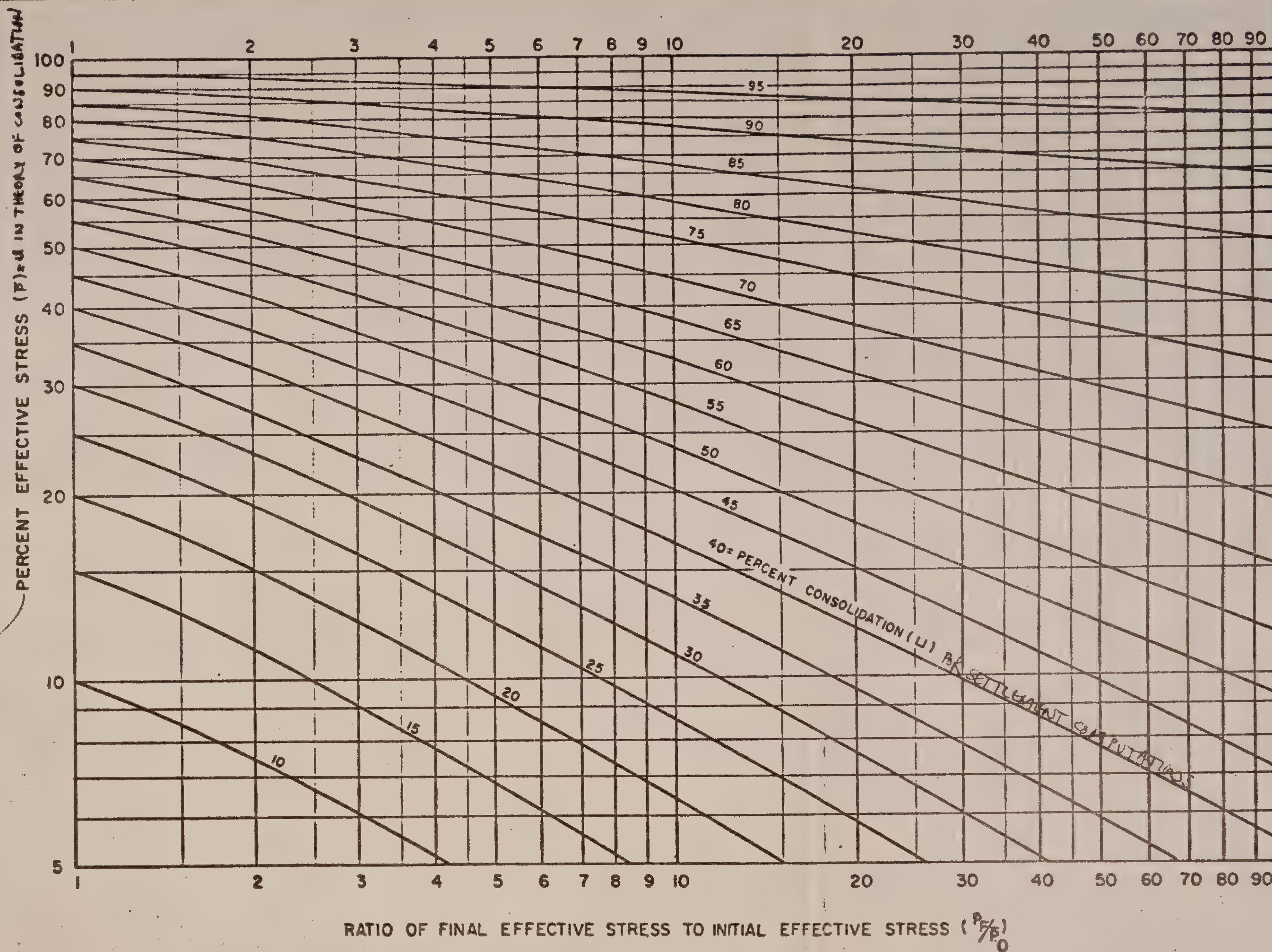
$$U = \frac{e_o - e_i}{e_o - e_f} = \frac{C_c \log \bar{p}_i/\bar{p}_o}{C_c \log \bar{p}_f/\bar{p}_o}$$

$$\log \bar{p}_i/\bar{p}_o = U \log \bar{p}_f/\bar{p}_o$$

$$\bar{p}_i/\bar{p}_o = \left(\bar{p}_f/\bar{p}_o\right)^U$$

$$\bar{P} = \frac{\bar{p}_i - \bar{p}_o}{\bar{p}_f - \bar{p}_o} = \frac{\bar{p}_i/\bar{p}_o - 1}{\bar{p}_f/\bar{p}_o - 1} = \frac{(\bar{p}_f/\bar{p}_o)^U - 1}{\bar{p}_f/\bar{p}_o - 1}$$

THE ABOVE EQUATION IS SHOWN IN GRAPHICAL FORM TO THE RIGHT.

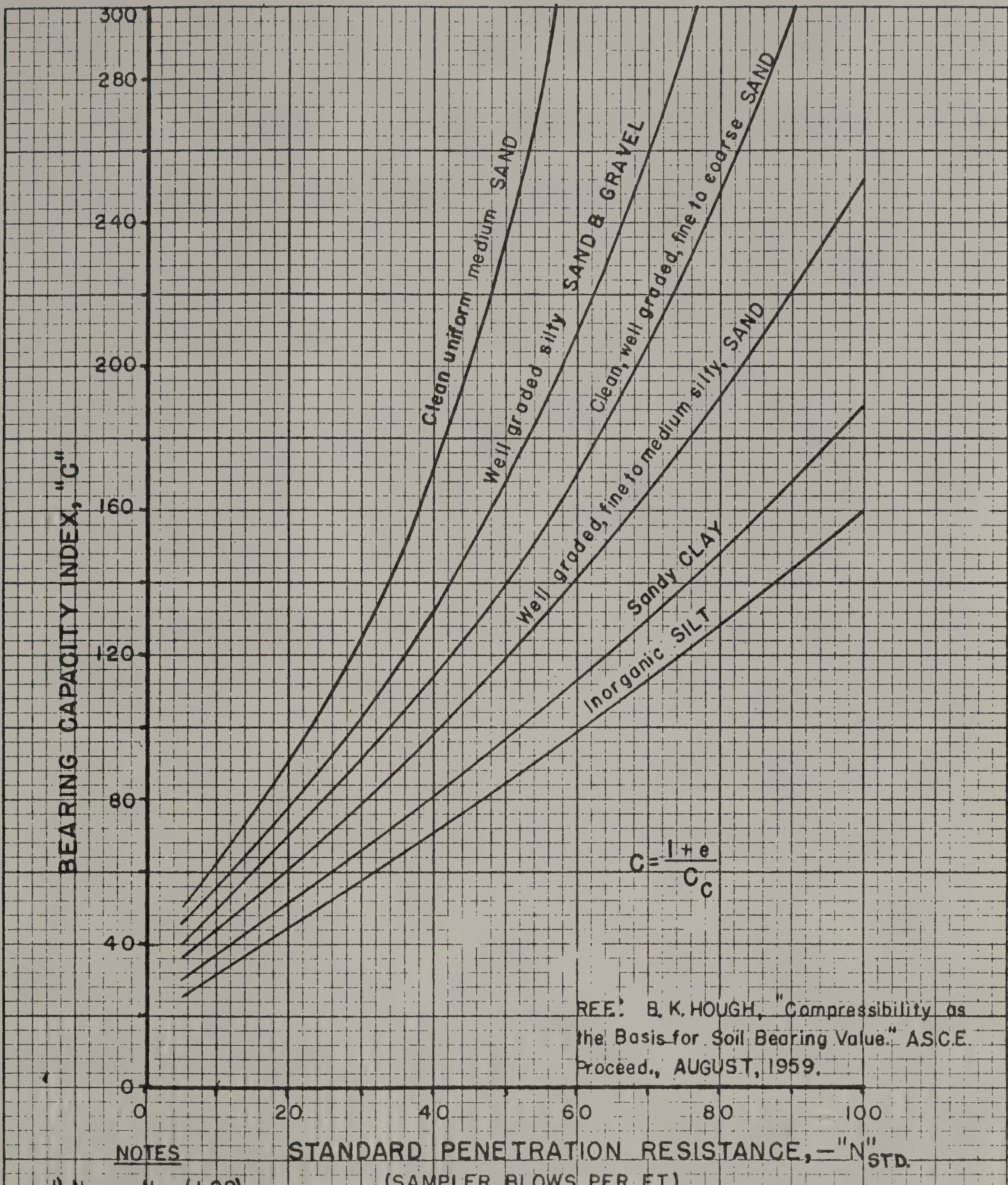


THE PERCENT GAIN IN STRENGTH OF A FOUNDATION SOIL IS DIRECTLY RELATED TO THE PERCENT NORMAL EFFECTIVE STRESS INCREASE (MOHR DIAGRAM). SINCE THE PERCENT EFFECTIVE STRESS IS NOT THE SAME AS THE PERCENT CONSOLIDATION, AS SHOWN IN THE ABOVE CHART, THE PERCENT CONSOLIDATION SHOULD NOT BE USED TO DETERMINE STRENGTH GAIN OR SAFETY FACTOR INCREASE WITH TIME AFTER LOADING. THE PERCENT CONSOLIDATION "U" AS USED IN THE COMPUTER STABILITY ANALYSES SHOULD BE PERCENT EFFECTIVE STRESS.

NOTE: ABOVE CHART GOOD ONLY FOR STRAIGHT LINE PORTION OF E LOG $\bar{P}$ PLOT.

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS DIVISION OF CONSTRUCTION BUREAU OF SOIL MECHANICS
RELATIONSHIP BETWEEN PERCENT CONSOLIDATION AND PERCENT EFFECTIVE STRESS

APPROVED 4/18/1967 Wm. P. Hofmann DIRECTOR	DISTRICT NO. COUNTY MO
	DWG NO SM 1691



NOTES

1) $N_{STD.} = N_{N.Y.}(1.29)$

2) $N_{STD.}$ – SAMPLER BLOWS FOR A 140LB. HAMMER FALLING 30 INCHES.

$N_{N.Y.}$ – SAMPLER BLOWS FOR A 300LB. HAMMER FALLING 18 INCHES.

3) FOR DENSE FULLY SATURATED FINE SANDS AND SILTS WITH SAMPLER BLOWS (N) GREATER THAN 15., MODIFY "N" AS FOLLOWS;

$$N' = 15 + 1/2\,(N - 15)$$

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DEPARTMENT OF PUBLIC WORKS
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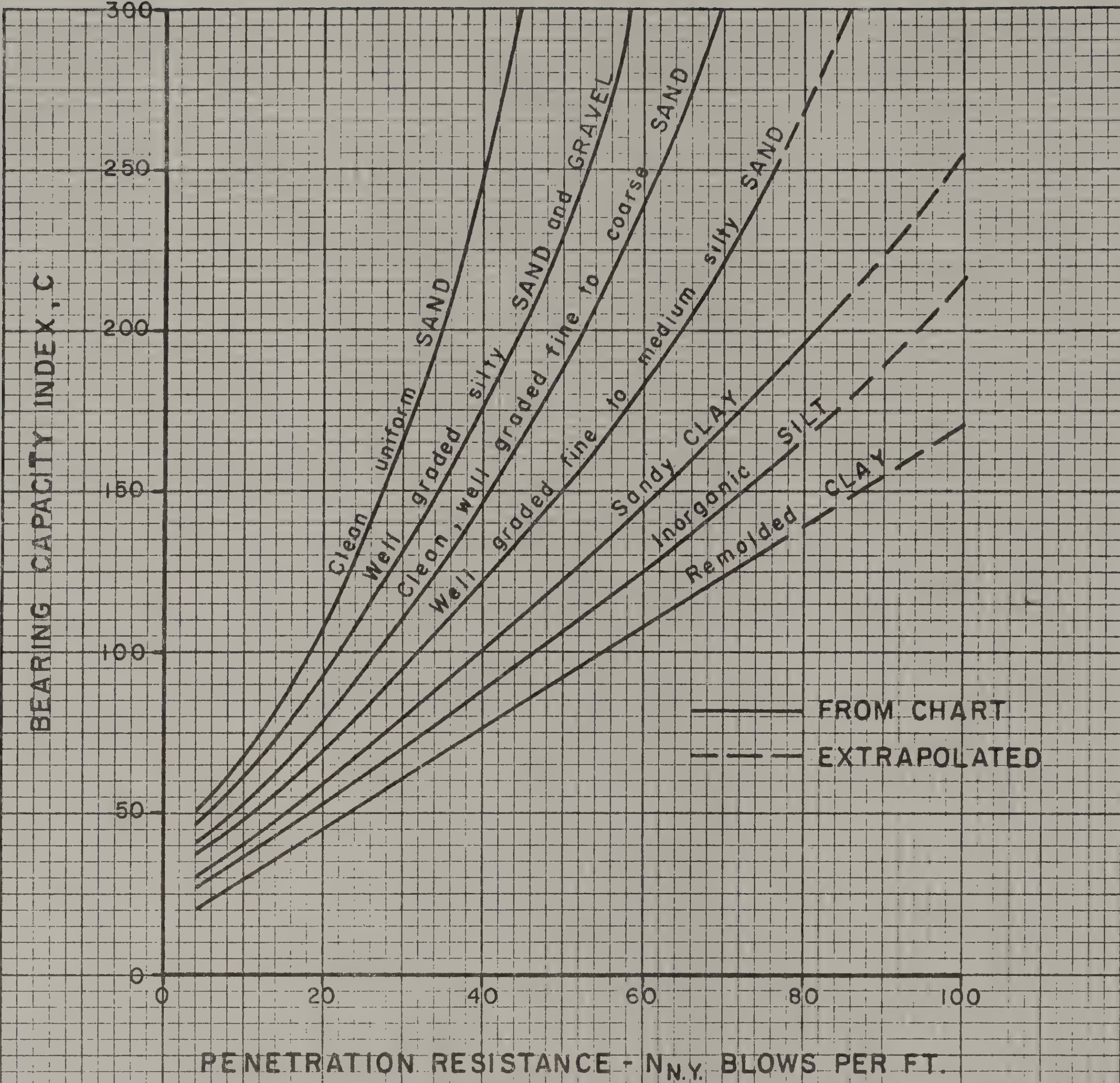
BUREAU OF SOIL MECHANICS

BEARING CAPACITY INDEX VALUES FOR GRANULAR SOILS

APPROVED 9/3 1963	DISTRICT NO. 1
W. P. Hofmann	COUNTY ALBANY
PRINCIPAL SOILS ENGINEER	DWG. NO. MO SM 1638



$N_{N.Y.}$ - SAMPLER BLOWS FOR A 300LB. HAMMER DROPPING 18 INCHES.

$C = \frac{1+e}{C_c}$

REF. B.K. HOUGH, "Compressibility as the Basis for Soil Bearing Value." A.S.C.E. Proceed., AUGUST, 1959

PREPARED BY D. N. Geoffroy

DRAWN BY J. J. Maio

CHECKED BY D. N. Geoffroy

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

BEARING CAPACITY INDEX VALUES FOR N.Y.S. ST'D. BLOWS

APPROVED 8/10 1964
Wm. P. Hofmann
PRINCIPAL SOILS ENGINEER

DISTRICT NO. 1
COUNTY ALBANY

DWG. NO MO SM 1658

MEMORANDUM
DEPARTMENT OF TRANSPORTATION

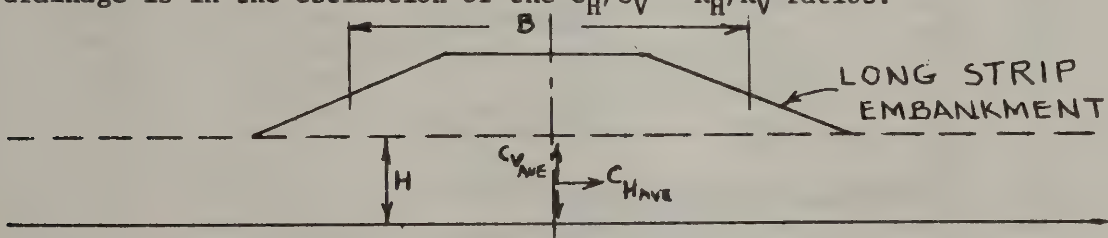
DATE December 20, 1976

SUBJECT RECOMMENDED PROCEDURES FOR INCLUSION OF LATERAL DRAINAGE INTO ANALYSIS OF TIME RATE OF CONSOLIDATION UNDER CENTERLINE OF LONG EMBANKMENT

FROM R. L. Gemme Raymond L. Gemme

TO V. C. McGuffey

As requested by you, I have conducted a literature search on the latest and apparently most practical methods of including lateral drainage into our time rate of settlement analyses. The need for this search is based on the fact that our previous method as shown on page 6-5 of our design manual is based on free lateral draining boundaries at the toes of the embankment slopes. This assumption in most cases underestimates the time of consolidation. It has been found that with the lateral boundary fixed at least at 5H from the embankment centerline, no noticeable effect on the rate of settlement occurred from the true lateral boundary condition of infinity. Of more importance the most difficult aspect of predicting rates of consolidation with lateral drainage is in the estimation of the $C_H/C_V = K_H/K_V$ ratios.



Method - From "Two Dimensional Consolidation of a Layer with Double Drainage Under a Strip Load" by S. M. Lacasse, M. F. Soulie, and C. C. Ladd (1975) Interim Report, Dept. of Civil Eng., Ecole Polytechnique, Montreal. Single drainage curves from "Consolidation of a Layer Under a Strip Load" by J. T. Christian and J. W. Boehmer (1972) ASCE, JSMFD Vol. 98, SM 7.

1. Compute the ratio of $t_{isotropic}/t_{anisotropic}$ from the following equation. t = TIME

$$\frac{t_{isotropic}}{t_{anisotropic}} = \propto \frac{T_V}{T_{2D}} \sqrt{C_H/C_V}$$

C_H/C_V :- C_H & C_V are average values for entire layer (See figures 1 & 2)
- See Table 1 for methods of determining this ratio
- C_V is determined from page 6-6 of our design manual

V. C. McGuffey
December 20, 1976
page 2

T_V= time factor from Terzaghi's theory for vertical drainage only

T_{2D} = time factor for two way drainage assuming isotropic permeability ($C_H = C_V$). Determine from Figure 3.
For one way drainage use T_{1D} from Fibure 4.

α = correction factor for ($C_H \neq C_V$). Determine from Figures 5 thru 5D.

2. If this ratio is less than 1 then increase it by the ratio of the Lacasse to Davis & Poulos solutions shown on Figure 6.

Example Problem - Double Drainage

$C_H/C_V = 2$, $B/H = 0.5$

U	T_V	T_{2D} (Fig. 3)	α (Fig. 5)	t_{ISO}/t_{ANIS} (Lacasse)
10	.008	.0013	.88	7.66
30	.071	.034	.85	2.51
50	.197	.130	.85	1.82
70	.403	.330	.85	1.47
85	.684	.750	.83	1.07

Example Problem - Single Drainage

$C_H/C_V = 2$, $B/H = 0.5$

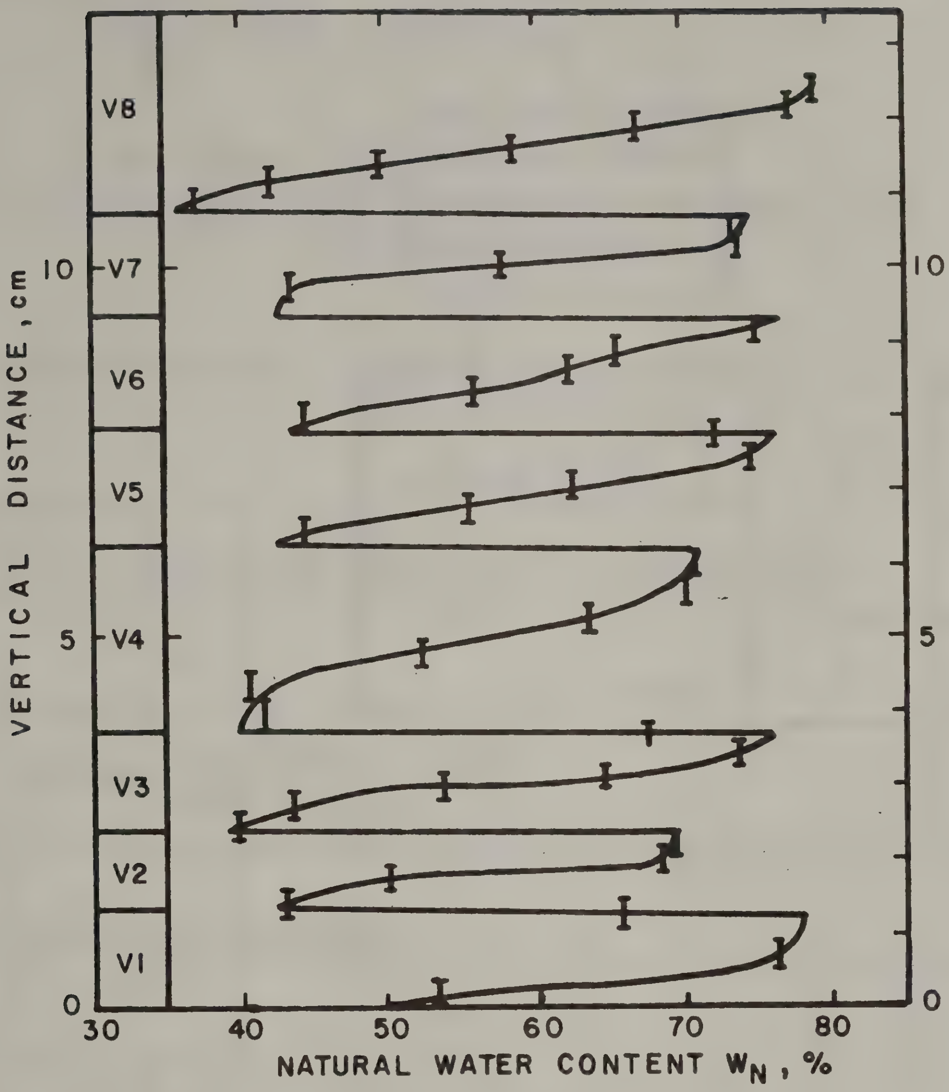
U	T_V	T_{1D} (Fig. 4)	α (Fig. 5)	t_{ISO}/t_{ANIS} (Christian)
10	.008	.007	$\boxed{.98}$	1.60
30	.071	.035	$\boxed{.97}$	2.78
50	.197	.09	.95	2.94
70	.403	.25	$\boxed{.90}$	2.05
85	.684	.46	.85	1.79

Note: Values in squares are assumed.

Comments

1. Values of α for other percentages of consolidation to my knowledge have not yet been published. The reason for this is perhaps the running times for these finite element programs are so large. We will add these parameters to our design charts as they become available from future publications.
2. These results will not function well if there are large undrained strains such as when the safety factor against a stability failure approaches 1.

RLG:MF



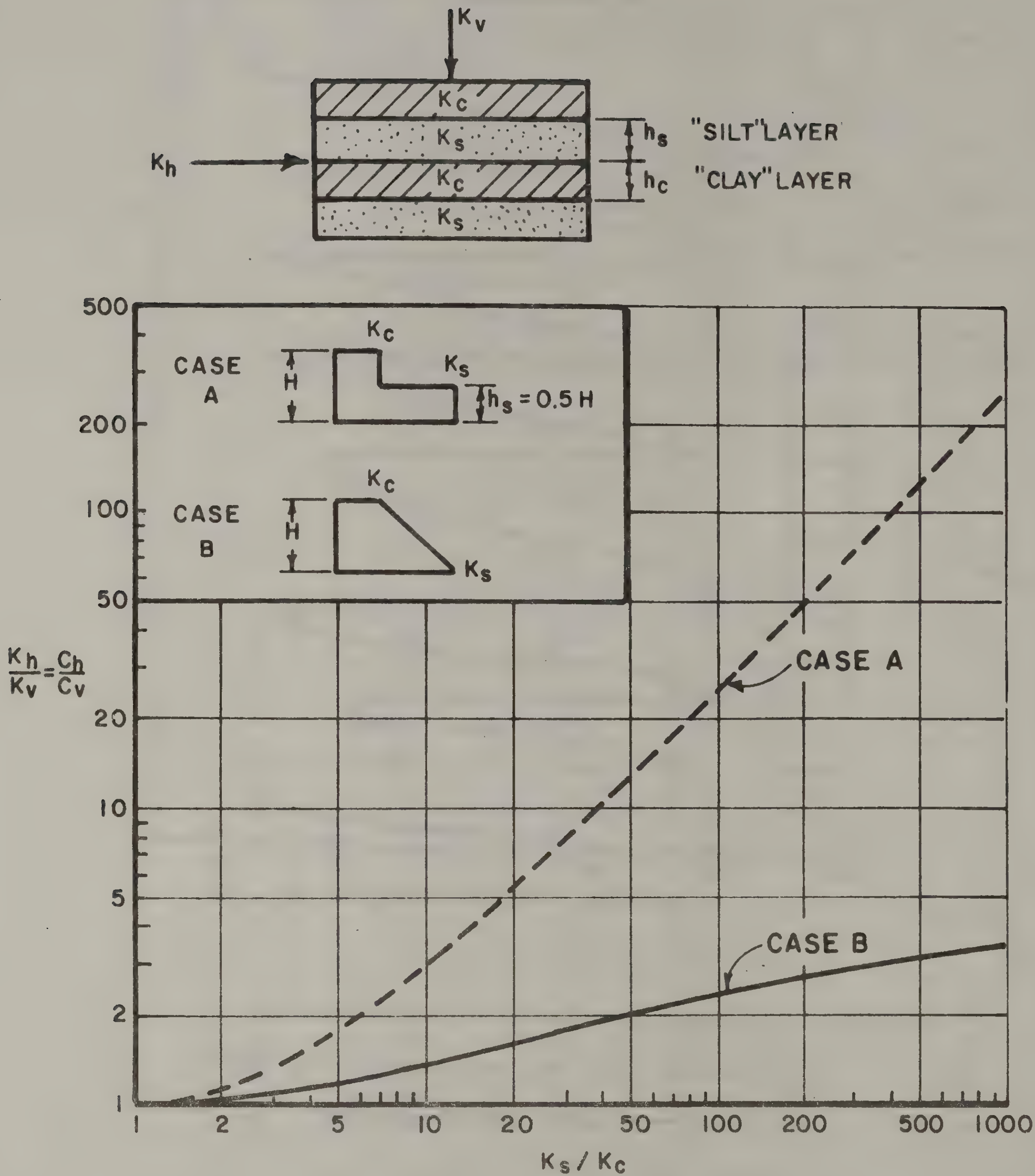
NOTES: (1) V1, V2, etc. refer to separate varves.

(2) Sample from Northampton, MA., W_N=56.7%

(3) Data from Ladd and Wissa (1970)

WATER CONTENT VARIATION WITHIN A VARVED CLAY FROM THE CONNECTICUT VALLEY

Figure 1



RELATIONSHIP BETWEEN K_h / K_v RATIO AND PERMEABILITY OF THE SILT AND CLAY LAYERS

(After Kenney, 1963)

Figure 2

METHOD OF DETERMINING AVERAGE COEFFICIENT OF CONSOLIDATION FOR LAYERED SYSTEMS

1) K FOR EACH LAYER : $K = \frac{a_v c_v \gamma_w}{1+e}$

2) K_v AVERAGE : $K_{AVE_V} = \frac{\Sigma H}{H_1/K_1 + H_2/K_2 + H_N/K_N}$

3) $(a_v/(1+e))_{AVE.} = \frac{H_1 \times a_1/(1+e_1) + H_2 \times a_2/(1+e_2) + H_N \times a_N/(1+e_N)}{\Sigma H}$

4) $C_{V_{AVE}} = \frac{K_{AVE_V}}{[a/(1+e)]_{AVE} \times 6.24} \left(\frac{IN^2}{MIN}\right)$

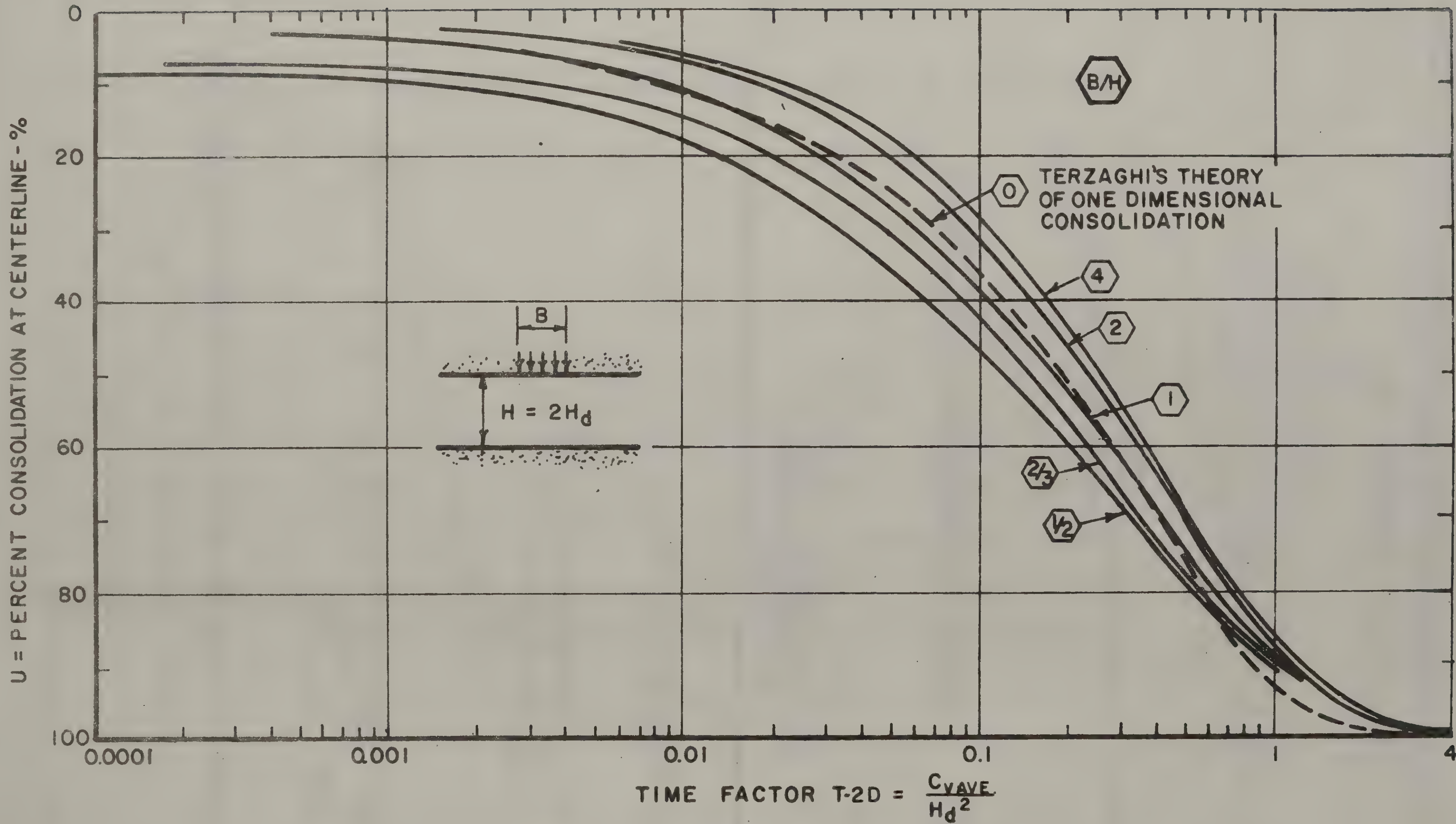
---

5) $K_{AVE_H} = \frac{H_1 K_1 + H_2 K_2 + H_N K_N}{\Sigma H}$

6) $C_{H_{AVE}} = \frac{K_{AVE_H}}{[a/(1+e)]_{AVE} \times 6.24} \left(\frac{IN^2}{MIN}\right)$

$\left(C_H/C_V\right)_{AVE} = \left(K_H/K_V\right)_{AVE}$ CASE A

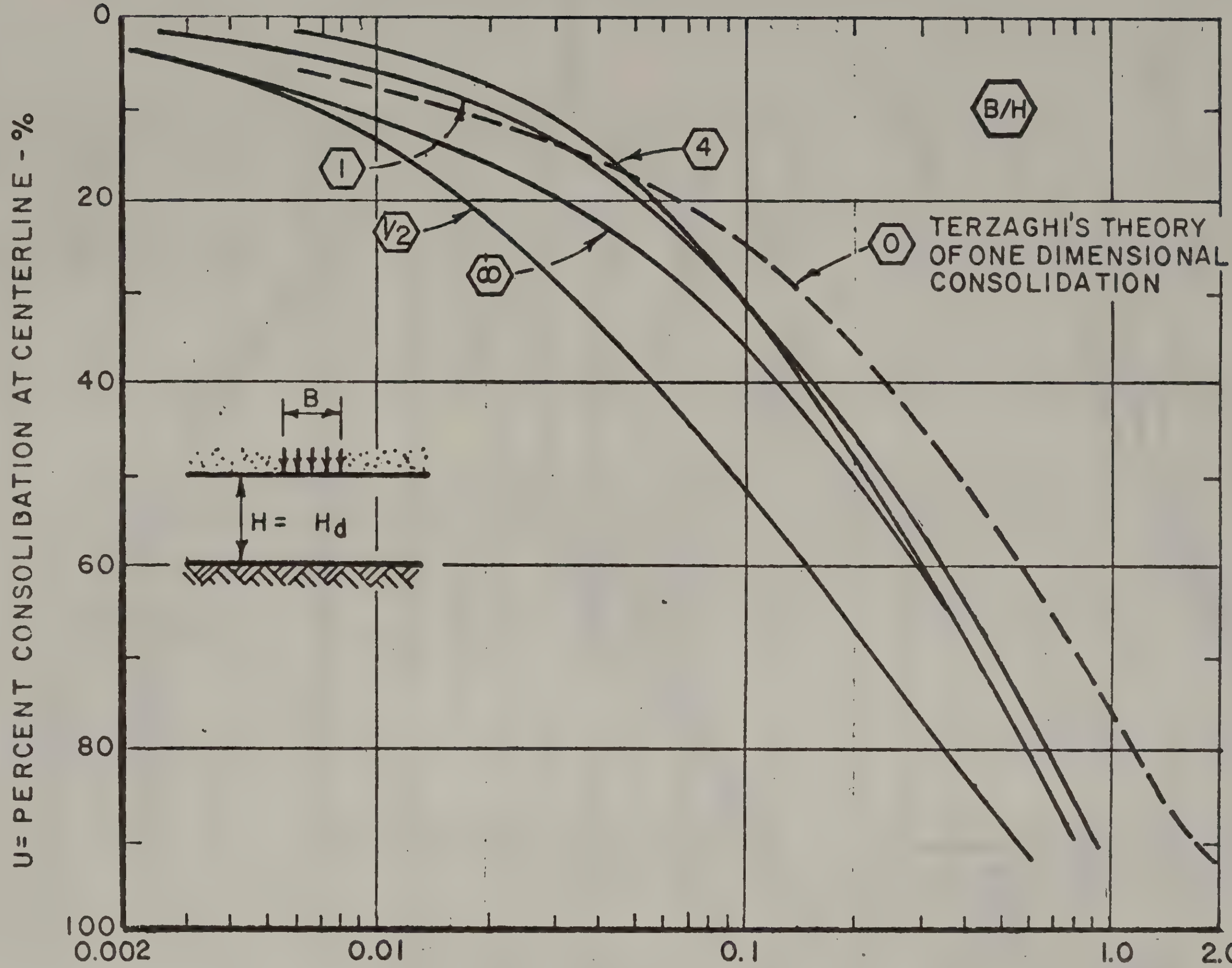
NOTE: THIS RATIO WILL BE MUCH LOWER IF CASE B APPLIES (SEE P. 6-8)



T-2D VS U FOR PLANE STRAIN CONSOLIDATION WITH DOUBLE DRAINAGE AND ISOTROPIC PERMEABILITY

(From Lacasse et al., 1975)

Figure 3



TIME FACTOR T-ID = $\frac{C_{V\,AVE}}{H_d^2}$

T-ID VS U FOR PLANE STRAIN CONSOLIDATION WITH SINGLE DRAINAGE AND ISOTROPIC PERMEABILITY

(From Christain et al., 1972)

Figure 4

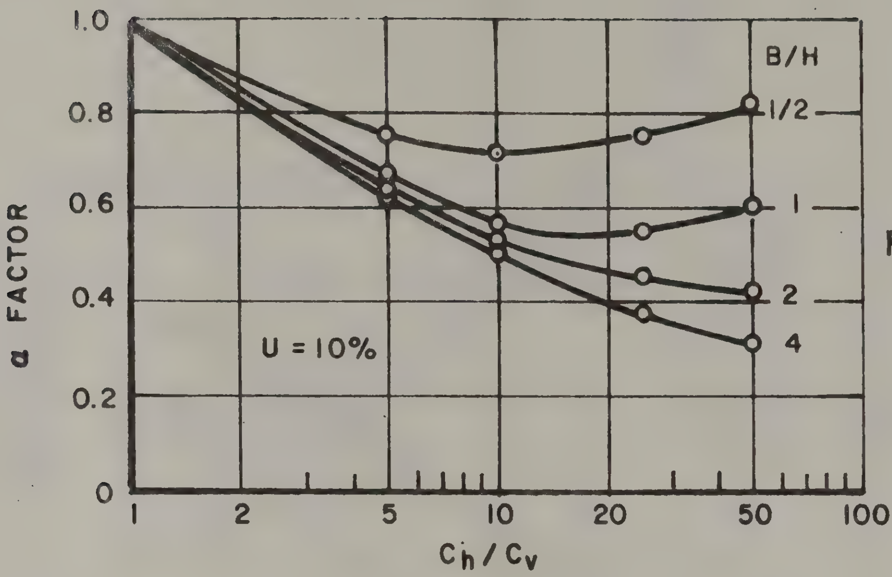


Figure 5

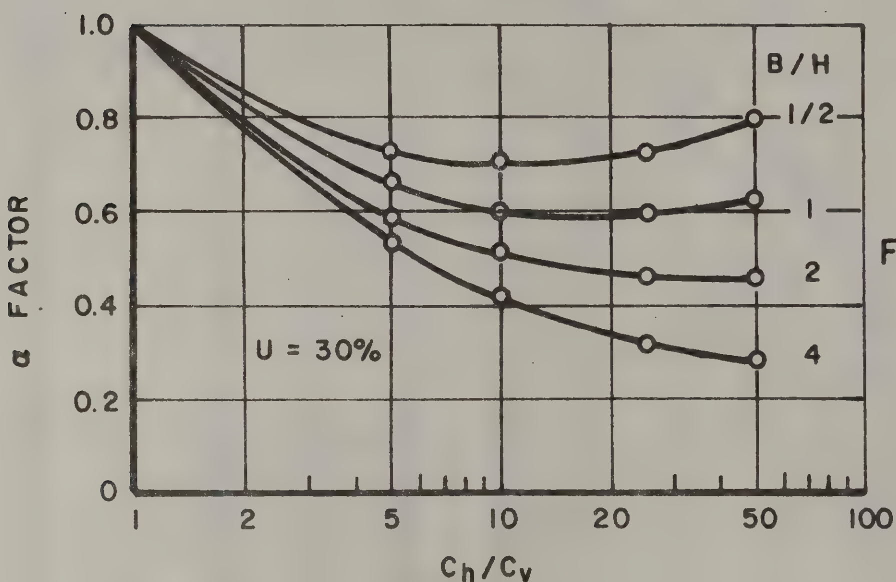
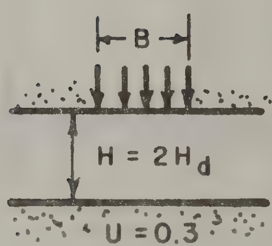


Figure 5A



$$t = (t_v)\left(\frac{T_{2D}}{T_v}\right)\left(\frac{1}{a\sqrt{k_h / k_v}}\right)$$

ALPHA FACTOR VS PERMEABILITY RATIO
PLANE STRAIN CONSOLIDATION
U = 10% and 30%
(From Lacasse et al., 1975), U=.3, Double Drainage

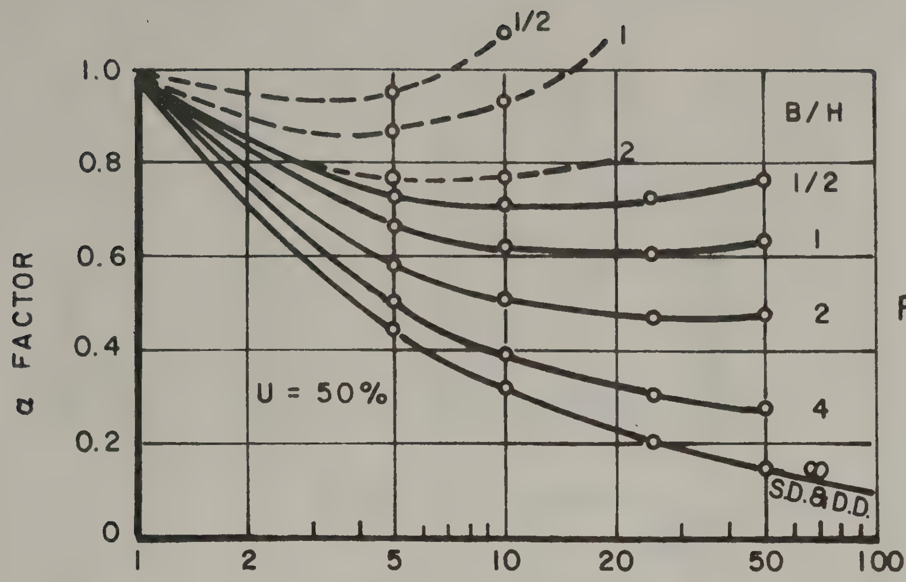


Figure 5B

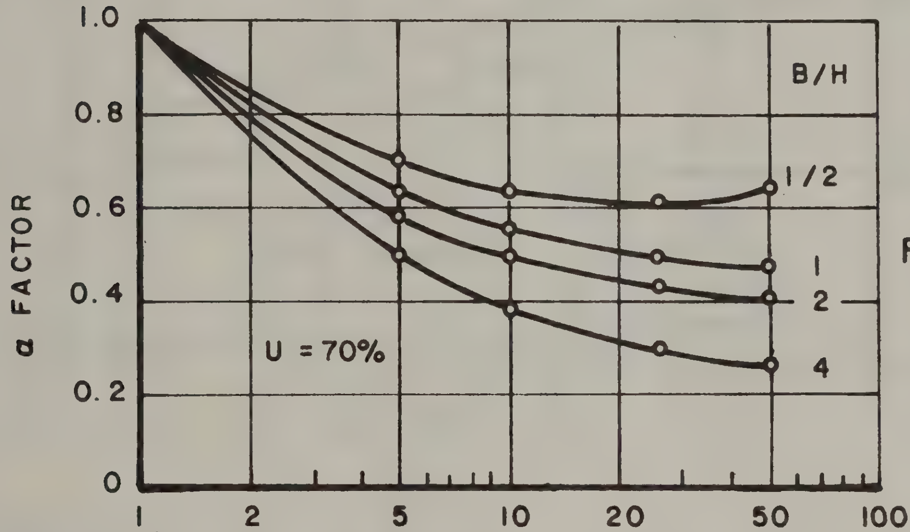


Figure 5C

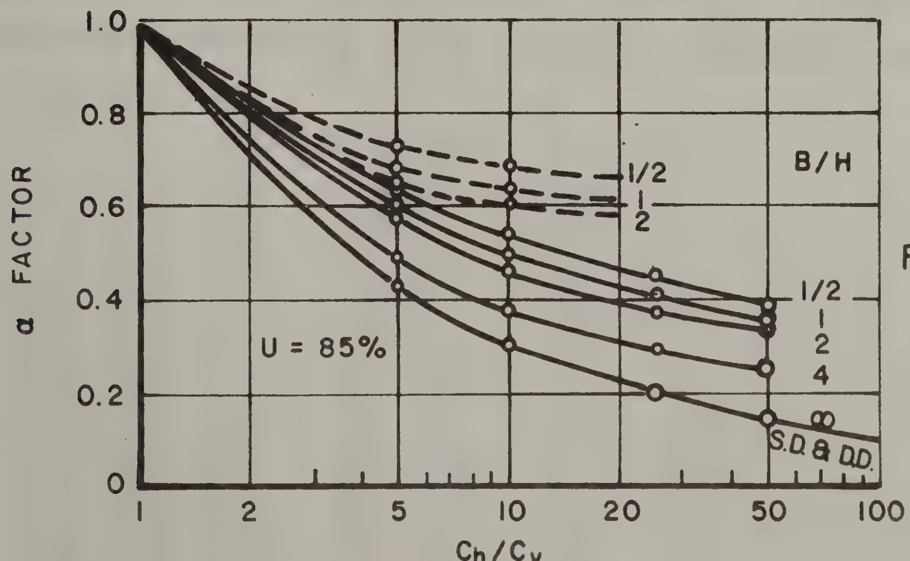
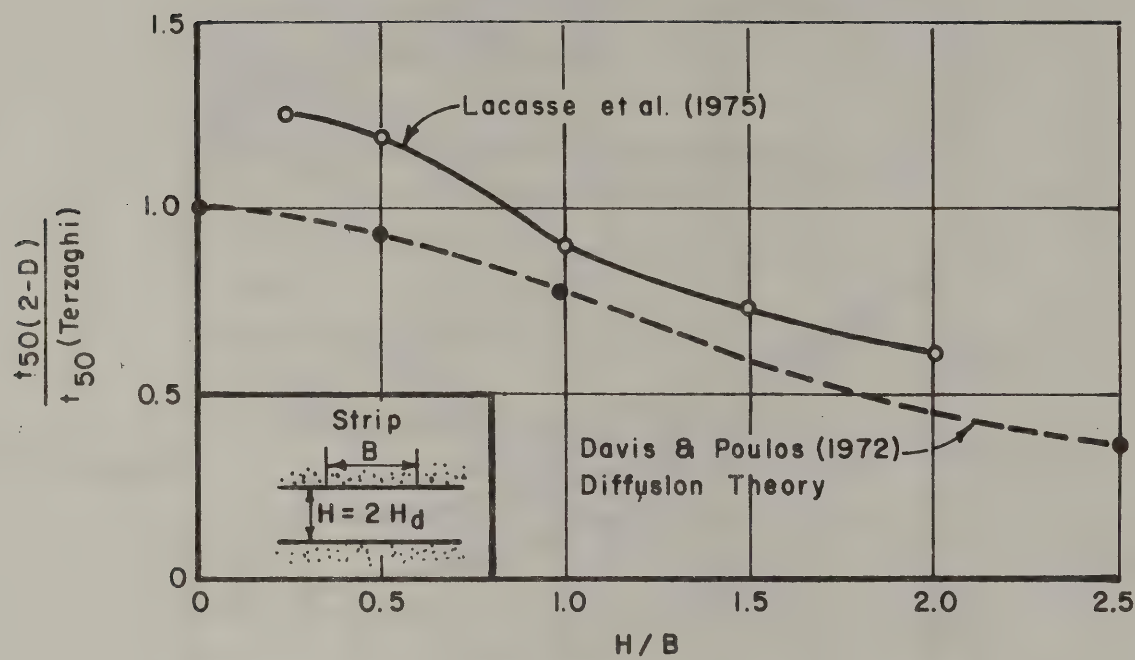


Figure 5D

——— (From Lacasse et al., 1975), U=.3, Double Drainage

– – – – (From Christian et al., 1972), U=.4, Single Drainage



(a) EFFECT OF LATERAL DRAINAGE ON RATE OF CONSOLIDATION FROM DIFFERENT THEORIES WITH ISOTROPIC PERMEABILITIES

Figure 6

TABLE 1

METHODS FOR MEASUREMENT OF C_h AND $K_h/K_v = C_h/C_v$

NO.	METHOD AND PARAMETER	REMARKS	REFERENCE
1	Laboratory consolidometer test on "horizontal" ($\theta = 90^o$) sample (C_h)	(1) Wrong m_v (2) Sample size influences results	Rowe (1959)
2	Laboratory consolidometer test with radial drainage to sides (C_h)	May have problems with side friction + scale effects	McKinley (1961)
3	Laboratory consolidometer test with radial drainage to vertical sand drain (C_h)	Large sample recommended to minimize scale effects	Rowe & Barden (1966) Shields & Rowe (1965)
4	Laboratory permeability tests on vertical and horizontal samples (C_h)	Problem with variability when using different samples	
5	Laboratory permeability tests on cubic sample (k_h/k_v)	Better than No. 4; large (10 cm) samples recommended	Chan & Kenney (1973) Ladd & Wissa (1970) Saxena et al. (1974)
6	Field constant head flow tests with hydraulic piezometer (c_h, k_h)	(1) Method of installation important (2) Need to consider length to diameter ratio	Mitchell & Gardner (1975) P. 284-2EC IN SITU MEASUREMENTS VOL. II
7	Field pumping test from vertical sand drain (K_h)	(1) Method of installation important (2) Pervious layers can have important effect	Casagrande & Poulos (1969)

Average consolidation for clay cylinders of outer diameter 2R drained radially to axial drain well of diameter 2r. Outer and end surfaces impervious.

For square spacing; ℓ distance center to center

$2R = 1.128\ell$

for triangular spacing; ℓ distance center to center

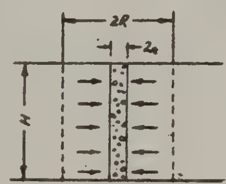
$2R = 1.05\ell$

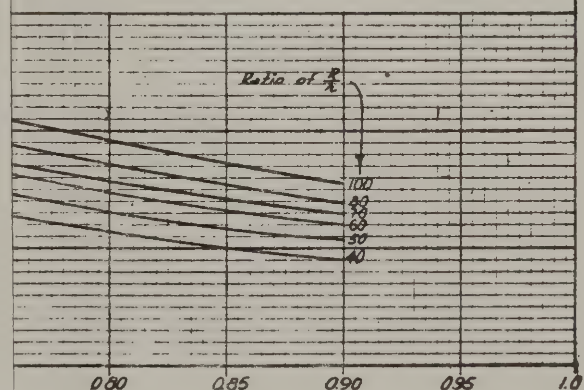
$$t_r = 4R^2 \frac{\delta w \cdot A_v}{K_h(1+e)} \cdot T_R = T_R \cdot \frac{4R^2}{C_{v_h}}$$

$$\text{if } \frac{K_h}{K_v} = n, \text{ then } \frac{C_{v_h}}{C_v} = n$$

$$\text{and } t_h = T_R \cdot \frac{4R^2}{nC_v}$$

$$\text{for } t_h = t_v;\ T_R = n \cdot \frac{H^2}{4R^2} \cdot T_v$$

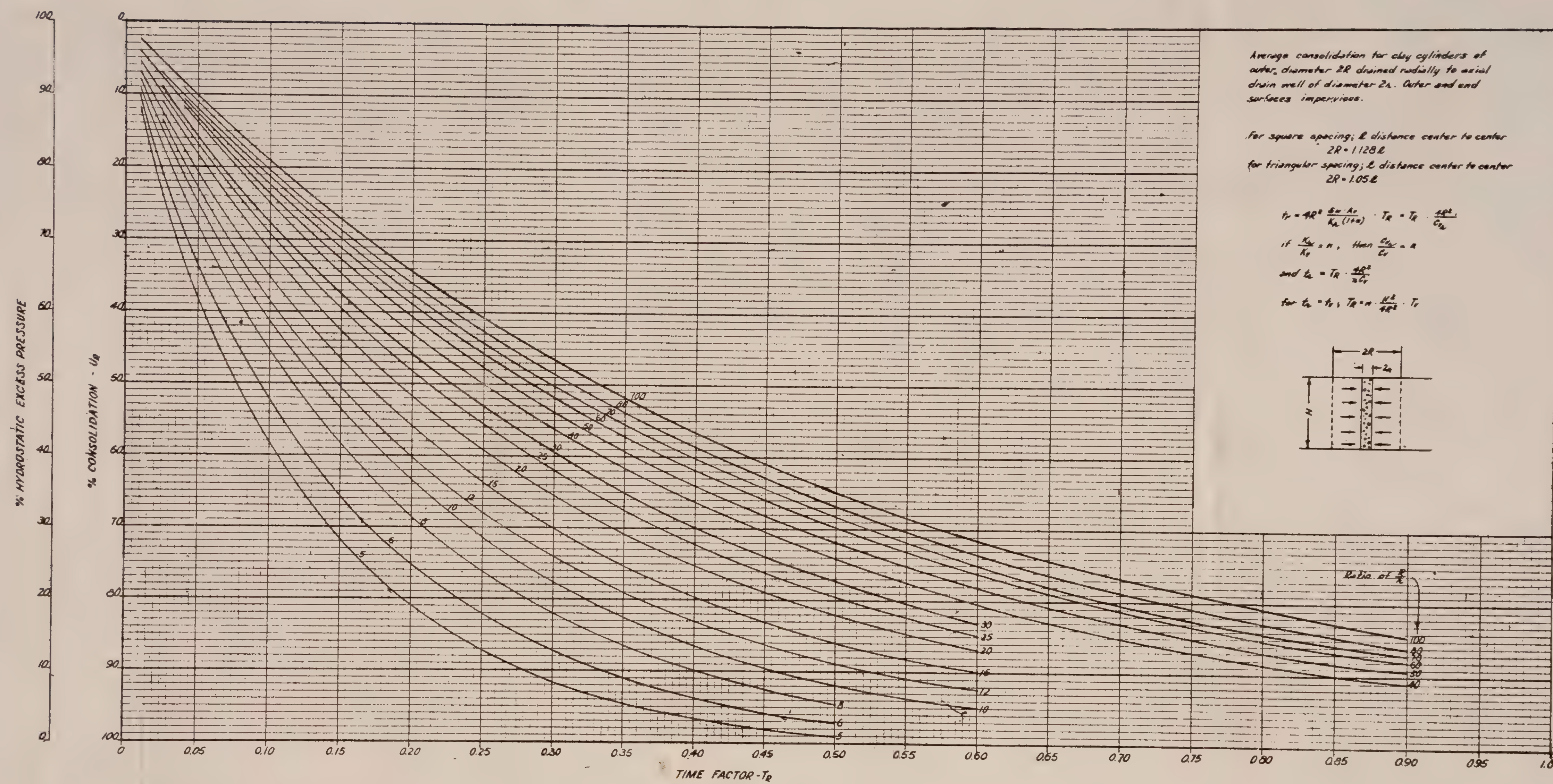




RELATION BETWEEN TIME FACTOR T_R AND DEGREE OF CONSOLIDATION U_R FOR CYLINDRICAL BODY DUE TO DRAINAGE TOWARD CENTER WELL FOR VARIOUS VALUES OF $\frac{R}{r}$

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS

DRAWING No. SM 571



These curves were plotted using data obtained from "The Influence of Drain Wells on the Consolidation of Fine Grained Soils.

RELATION BETWEEN TIME FACTOR T_R AND DEGREE OF CONSOLIDATION U_R FOR CYLINDRICAL BODY DUE TO DRAINAGE TOWARD CENTER WELL FOR VARIOUS VALUES OF $\frac{R}{r}$

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS

N.W.J. 7/29/48

DRAWING No. SM 571

6-16

fined settlement, as described in Chap. 8. For uniform deposits the settlement is found for each stratum, using the average properties of the soils determined by the laboratory tests. For variable-regular deposits the same procedure is used for each zone. For variable-irregular soils the settlements are found for both the poorest and the best soil conditions (or poorest and average), since it is the greatest differential settlement between columns which causes the most damage.

Ordinarily, the foundation settlements are computed only for representative parts of the structure: the center, the edge, and the corners of uniformly loaded structures; the largest, smallest, and typical columns of irregularly loaded structures; and for each critical unit, such as the foundations for heavy machines. If the soil conditions are variable, the settlements must be computed for more points than if the soils are uniform.

TABLE 6-10. LIMITING SETTLEMENTS (55-60)

Type of movement	Limiting factor	Maximum settlement
Total settlement	Drainage	6 to 12 in.
	Access	12 to 24 in.
	Probability of nonuniform settlement:	
	Masonry walled structure	1 to 2 in.
	Framed structures	2 to 4 in.
	Smokestacks, silos, mats	3 to 12 in.
Tilting	Stability against overturning	Depends on height and width
	Tilting of smokestacks, towers	$0.004b$
	Rolling of trucks, etc.	$0.01L$
	Stacking of goods	$0.01L$
	Machine operation—cotton loom	$0.003L$
	Machine operation—turbogenerator	$0.0002L$
	Crane rails	$0.003L$
	Drainage of floors	0.01 to $0.02L$
Differential movement	High continuous brick walls	0.0005 to $0.001L$
	One-story brick mill building, wall cracking	0.001 to $0.002L$
	Plaster cracking (gypsum)	$0.001L$
	Reinforced-concrete building frame	0.0025 to $0.004L$
	Reinforced-concrete building curtain walls	$0.003L$
	Steel frame, continuous	$0.002L$
	Simple steel frame	$0.005L$

NOTE: L = distance between adjacent columns that settle different amounts, or between any two points that settle differently. Higher values are for regular settlements and more tolerant structures. Lower values are for irregular settlements and critical structures.

FROM LEONARDS
"FOUNDATION ENGINEERING"

6-17

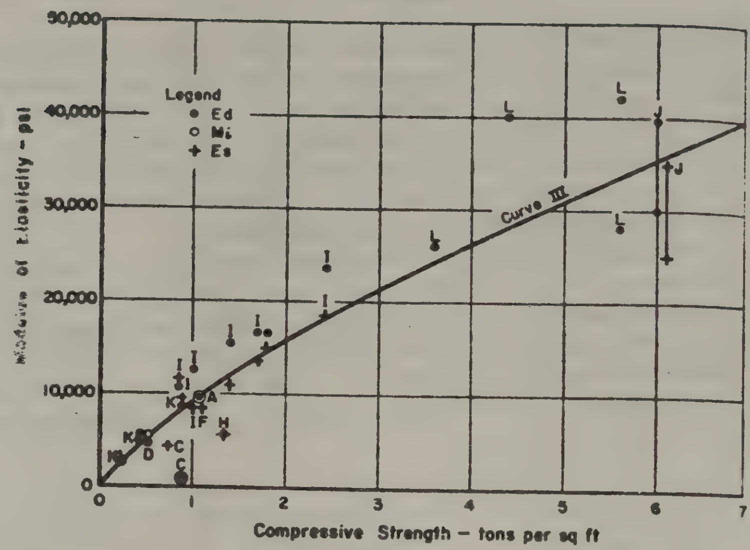


Fig. 9. Relationship of Modulus of Elasticity to Compressive Strength in Clays

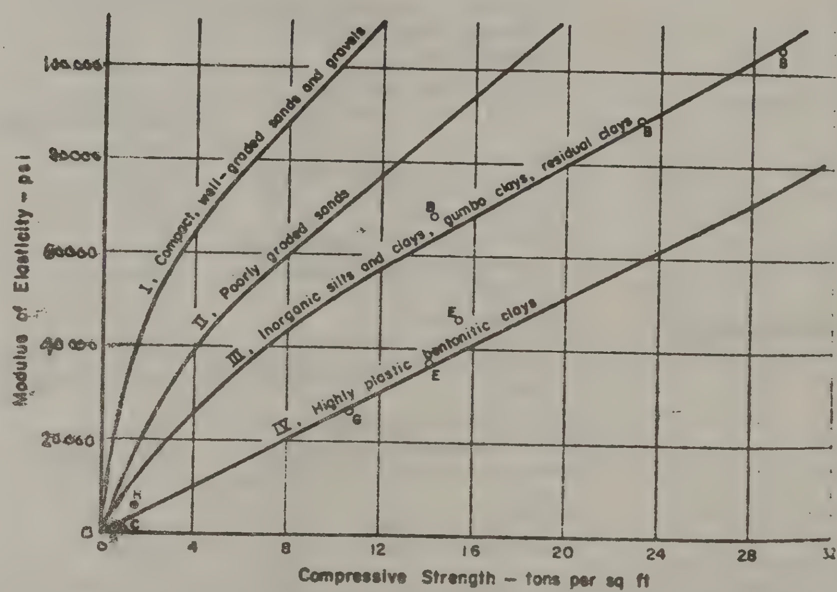
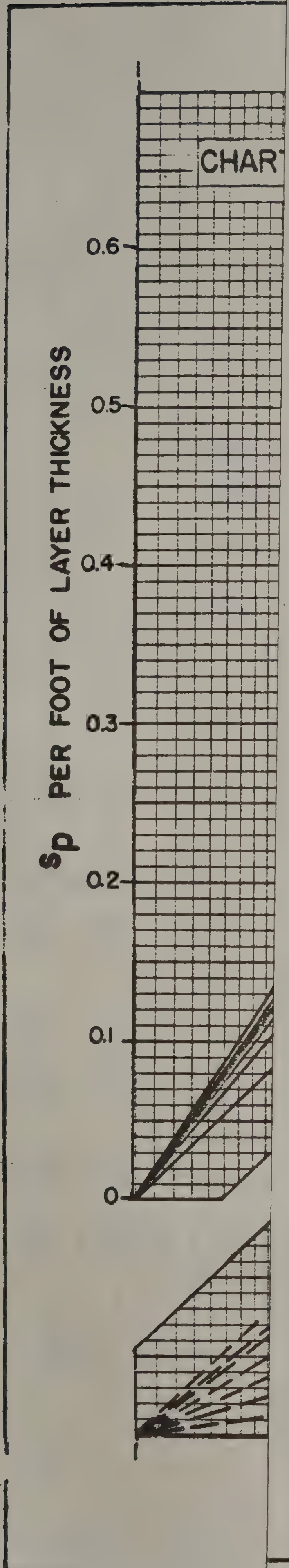


Fig. 10. Relationship of Modulus of Elasticity to Compressive Strength

$E \approx 1200 S_u$ (CLAYS)



SETTLEMENT PROPERTIES

1. COMPRESSION INDEX, C_c.

$$C_c = \frac{\text{MC in \%}}{100}$$

2. INITIAL VOID RATIO, e_0

MC(%)	e_0
100	2.2
200	4.2
300	6.0
400	8.0
500	10.0
600	11.6
700	13.0

3. COEFFICIENT OF SECONDARY CONSOLIDATION, C_s.

MC(%)	C_s
100	0.05
200	0.10
300	0.15
400	0.20
500	0.25
600	0.27
700	0.30

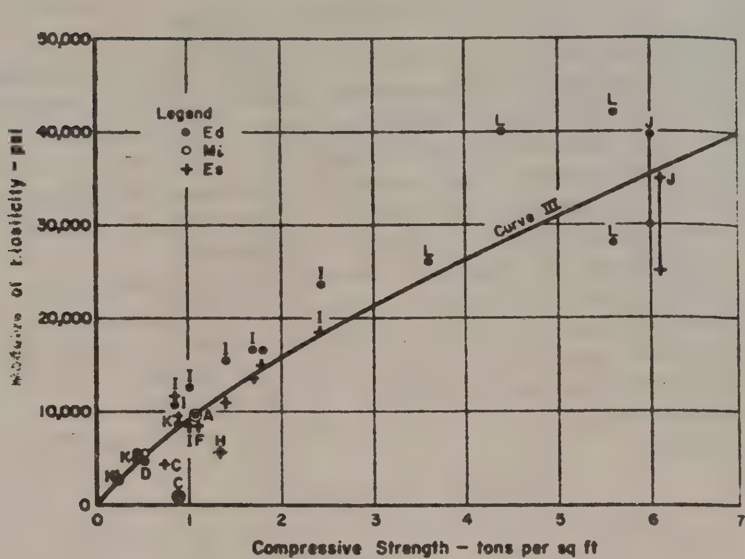
4. IT WAS ASSUMED THAT PRIMARY SETTLEMENT WILL BE COMPLETED IN ONE YEAR. MAKE NECESSARY COMPUTATIONS FOR TIME PERIODS LESS THAN 1 YEAR.

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

ESTIMATING SETTLEMENTS
IN
ORGANIC SOILS

APPROVED 9/2 1963 Wm. P. Hofmann PRINCIPAL SOILS ENGINEER	DISTRICT NO. 1 COUNTY ALBANY
	DWG. NO MOSM 1639

6-17



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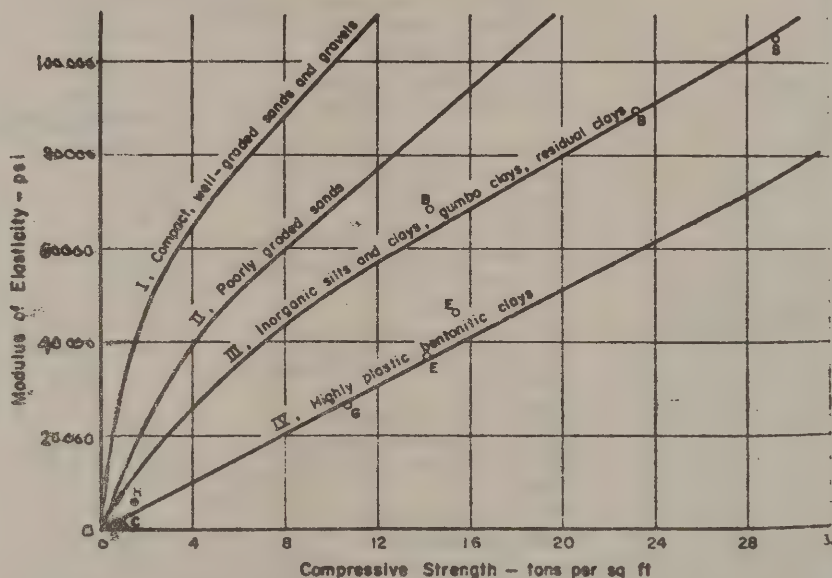

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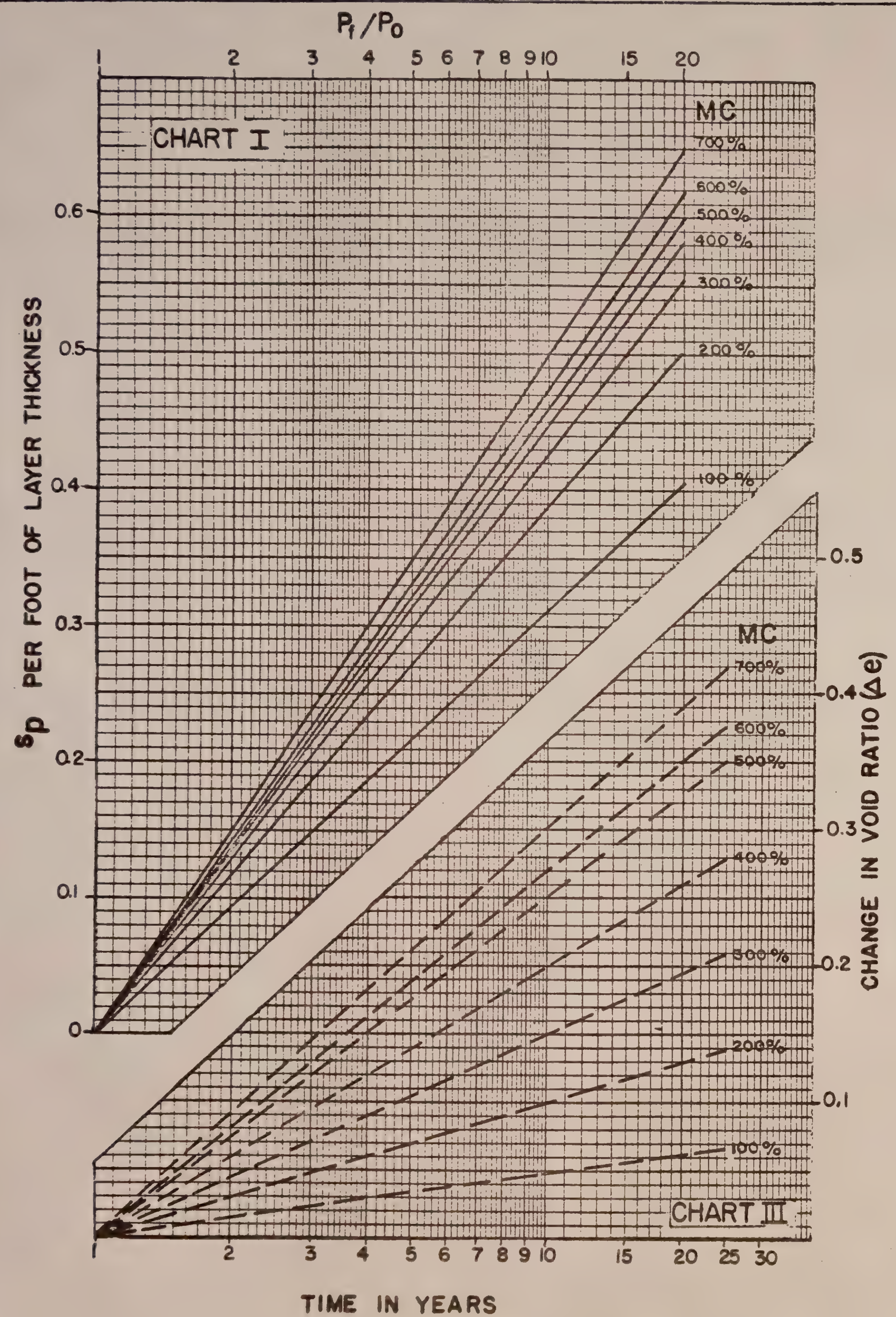
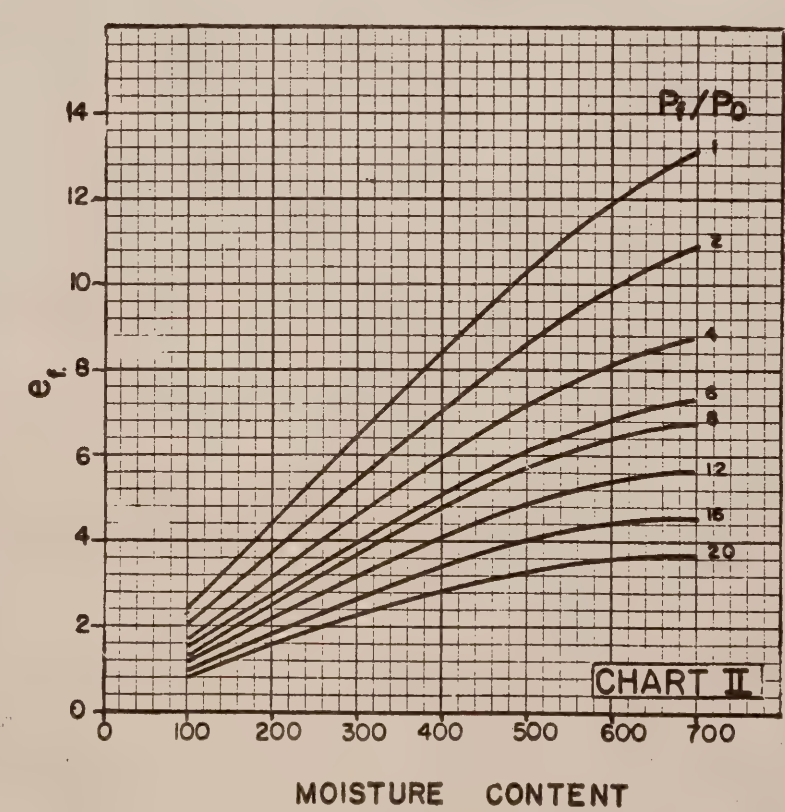


PROCEDURE

1. ENTER CHART I WITH THE P_f/P_o RATIO AND MOISTURE CONTENT, DETERMINE s_p.
2. ENTER CHART II WITH THE P_f/P_o RATIO AND MOISTURE CONTENT, DETERMINE e_f.
3. ENTER CHART III WITH MOISTURE CONTENT AND THE TIME, DETERMINE Δe.
4. THE AMOUNT OF TOTAL SETTLEMENT IS COMPUTED USING THE FOLLOWING EQUATION:

$$S = s_p H + \frac{\Delta e}{1+e_f} H$$

WHERE:
S = TOTAL SETTLEMENT
H = TOTAL LAYER THICKNESS
s_p = THE AMOUNT OF PRIMARY SETTLEMENT IN A ONE FOOT LAYER
$\frac{\Delta e}{1+e_f}$ = THE AMOUNT OF SECONDARY SETTLEMENT IN A ONE FOOT LAYER FOR A GIVEN TIME



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STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

ESTIMATING SETTLEMENTS IN ORGANIC SOILS

APPROVED 9/2 1963 Wm. P. Hofmann
PRINCIPAL SOILS ENGINEER

DISTRICT NO. 1
COUNTY ALBANY
DWG. NO MO-SM 1639

SECTION 7

STABILITY ANALYSIS

PAGES

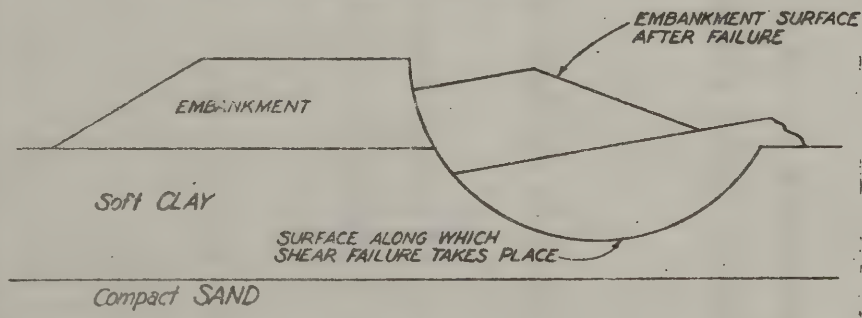
7-1	OUTLINE OF EMBANKMENT STABILITY ANALYSIS PROCEDURE
7-2 TO 7-4	OVERSTRESS ANALYSIS ON SENSITIVE CLAYS
7-2A TO 7-2E	STABILITY ANALYSIS - NYS CIRCULAR ARC GRAPHICAL ANALYSIS
7-5	STABILITY ANALYSIS - MODIFIED SWEDISH GRAPHICAL ANALYSIS
7-6 & 7-7	CONCEPTS & INFORMATION REQUIRED FOR CUTSLOPE DESIGN
7-8 & 7-9	" " " " FILL DESIGN

MANUALS

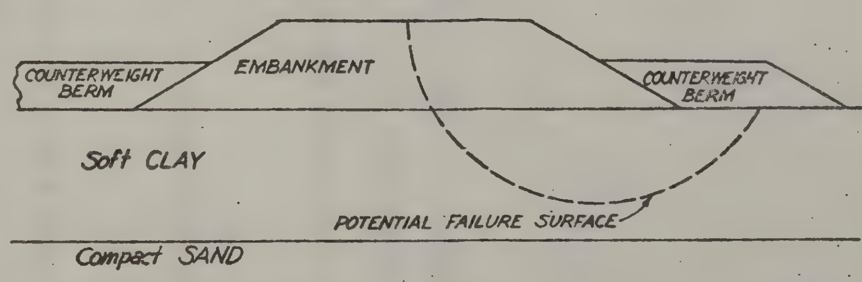
SEM 4/74	STABILITY CURVES FOR EMBANKMENTS ON SOFT SOILS
SEM 5/74	COMPUTERIZED MODIFIED SWEDISH WEDGE STABILITY ANALYSIS
SEM 3/72	INFINITE SLOPE ANALYSIS
SEM 1/72	BISHOP SLOPE STABILITY ANALYSIS BY DESK TOP COMPUTER
SEM 2/72	NAVDOCKS WEDGE ANALYSIS BY DESK TOP COMPUTER
SDP-1 10/70	COMPUTERIZED ANALYSIS OF THE STABILITY OF EARTH SLOPES

EARTH CUTSLOPE DESIGN IN NEW YORK STATE, J.C. McGUFFEY 1976

CROSS-SECTION OF HIGHWAY EMBANKMENT SHOWING SHEAR FAILURE



CROSS-SECTION OF HIGHWAY EMBANKMENT SHOWING COUNTERWEIGHT BERMS TO PREVENT FAILURE



DETERMINATION OF STABILITY PROBLEM

1. EXPLORATIONS
   - A. SUFFICIENT TO DEVELOP DEPTH AND AREA EXTENT OF SOFT SOILS
   - B. UNDISTURBED SAMPLE DRILL HOLES FOR LABORATORY TESTING
2. LABORATORY TEST PROGRAM
   - A. STRENGTH TEST ON UNDISTURBED SAMPLES
   - B. CLASSIFICATION TESTS TO CORRELATE STRENGTH RESULTS WITH SOIL FROM OTHER AREAS INVESTIGATED BY "DISTURBED" SAMPLING METHODS
3. DESIGN PROGRAM
   - A. STABILITY ANALYSIS

$$\text{SAFETY FACTOR FOR STABILITY} = \frac{\text{RESISTING MOMENT (FROM STRENGTH OF SOIL ALONG ARC)}}{\text{OVERTURNING MOMENT (FROM WEIGHT OF FILL)}}$$

   - B. DETERMINE BEST METHOD OF TREATMENT

METHODS OF SOLVING STABILITY PROBLEMS

1. CHANGE LOAD LOCATION TO AREAS OF BETTER SOIL CONDITIONS
2. DECREASE HEIGHT OF EMBANKMENT
3. COUNTERWEIGHT BERMS
4. EXCAVATION OF SOFT SOIL
   - A. TOTAL EXCAVATION
   - B. PARTIAL EXCAVATION
5. DISPLACEMENT OF SOFT SOIL
6. SLOW-RATE CONSTRUCTION
7. USE OF LIGHT WEIGHT MATERIAL FOR EMBANKMENT
8. VERTICAL SAND DRAINS

FACTORS THAT DETERMINE SELECTED METHOD

1. PROPERTIES OF SOIL
   - A. SOIL PROFILE
   - B. SHEARING STRENGTH CHARACTERISTICS
   - C. RATE OF CONSOLIDATION AND RELATED STRENGTH INCREASE
2. ECONOMICS
   - A. AVAILABLE MATERIALS - QUANTITY AND COST
3. EMBANKMENT DIMENSIONS
4. CONSTRUCTION SCHEDULE
5. RIGHT-OF-WAY

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

OUTLINE OF EMBANKMENT STABILITY ANALYSIS PROCEDURES

APPROVED 19	DISTRICT NO. MO COUNTY
PRINCIPAL SOILS ENGINEER	DWG. NO 45 SM 1614

RAYMOND L GEMME 7-2

MEMORANDUM
DEPARTMENT OF TRANSPORTATION

DATE March 10, 1976

SUBJECT OVERSTRESS ANALYSIS ON SENSITIVE CLAYS

FROM R. L. Gemme

TO V. C. McGuffey

As we discussed, I have prepared the following guide to running overstress stability analysis on sensitive clays.

Overstress Stability Analysis

Applicability

Progress overstress analysis when clay is sensitive and shearing stress at any point exceeds shearing strength.

Procedure - 1. Check for Clay Sensitivity

- High moisture content clays with liquid limit exceeding moisture content by at least 10%.
- Unusually sharp break in the shear stress-strain curve.
- Ultimate strength is less than 80 percent of peak strength.
- Larger than normal pore pressure build-up during shear strength test.

2. Check for Overstress

- Plot shear stress with depth from chart on page 5-12 of design manual and compare to shearing strength with depth. The zone of overstress at time equals zero after embankment construction is where the shearing stress exceeds the shearing strength. This zone of overstress increases with time due to loss of shearing strength (ultimate strength) in the overstressed zone. The loss in strength continues until the net difference between the shearing stress and ultimate shearing strength is equal to zero (see Fig. No. 1) i.e. where

Area  = Areas  + 

From experience ultimate shearing strength has been found to be in the order of 0.75 times the maximum shearing strength for sensitive New York clays. The limits of overstress increase with time as indicated above from undrained stress distribution. However consolidation and strength gain also increase with time depending on the rate of embankment construction and soil coefficients of consolidation. This tends to decrease the zone of overstress with time. These two rate processes are opposite in effect. To complicate matters comparatively large strains occur in the overstressed zone causing decrease in permeability which affects consolidation and ultimate strength gain.

Since sensitive soils are usually very slow consolidating and it is difficult to determine ultimate strength gain effects, strength increase due to consolidation should be neglected in the final analysis.

3. Progress overall stability analyses by the Bishop method utilizing design shear strength as obtained in Figure No. 1.

References, Maximum Shearing Stress Charts =

1) BSM Design Book Pg. 5-12
2) Plastic Charts, BSM Foundation Design Section Files
3) Navdocks pg. 7-5-1
4) Burmister Charts, HRB, Proceedings of 35th Annual Meeting, 1956
6) The Application of Theories of Elasticity and Plasticity to Foundation Problems. Leo Jurgenson, Journal of the Boston Society of Civil Engineers, July, 1934.
7) Poulos, H.G. & E.H. Davis "Elastic Solutions for Soil and Rock Mechanics," John Wiley & Sons, 1974.

RLG:MF

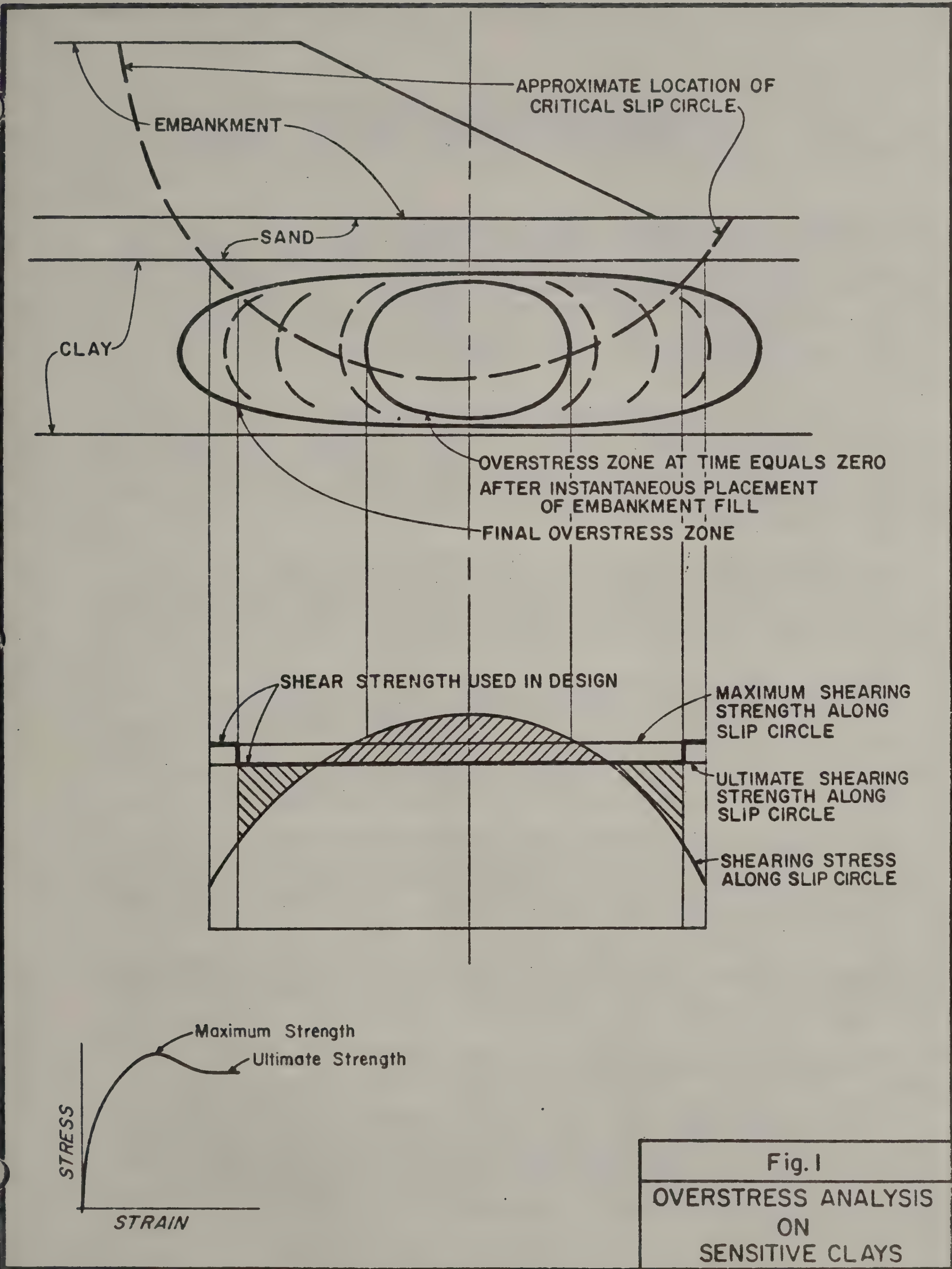


Fig. I

OVERSTRESS ANALYSIS ON SENSITIVE CLAYS

Stability Analysis of Layered Systems Using the Circular Arc Method

Introduction: From classroom discussions, it was pointed out that the "Circular Arc Method" is a tedious undertaking for a stability analysis. It was further concluded that Taylor's method gave rapid results for slopes of homogeneous soils and for the conditions it covered, the analysis itself was quite accurate. Naturally, the success or accuracy of any method of analysis depends almost in entirety upon an accurate estimation of the friction angle and cohesion of the mass.

However, it is immediately evident that Taylor's method is in most cases entirely inapplicable to slopes composed of fill material. This analysis is also no longer of any value except as an approximate check where the cross section of the slope is composed of distint layers of different materials, each layer having different physical properties.

For these reasons which make Taylor's method unsatisfactory in most analyses in New York State, the New York State Bureau of Soil Mechanics has adopted a modification of the "Circular Arc Method" which overcomes to their satisfaction the deficiencies of Taylor's method.

It is conceded that Taylor's method is more accurate, but then this method is only applicable for highly idealized conditions. This is not to infer that the method to be discussed here is not accurate since this is not true. The test of any analysis is the stability of the slopes in the field. The modified "Circular Arc Method" to be described here has passed this test very satisfactorily.

The discussion which follows is by no means complete to the smallest detail in theory or in practical aspects. However, it is intended to give the reader an introduction and working knowledge of the method with the hope that with use, an intimate understanding of it will be acquired.

The Method - Before going into any details, some general statements should be made to further illustrate the value of this method and other facts concerning it.

First of all, this method is purely graphical, as will be seen later.

Some of the more important basic assumptions are as follows. First, failure takes place along a circular arc about center. Each layer of the system has a determined average c and ϕ. Each layer of the system is consolidated under its own weight. Naturally these assumptions preclude the assumptions that the cross-section is accurately known.

Some of the complexities in slope stability analyses that can be accounted for using this method are sloping water tables variable densities and strengths as mentioned previously, artesian pressures, degrees of consolidation and irregular ground surfaces.

In Figs. I and IA, a complete analysis is illustrated which shows the considerations and mechanics of the method. For simplicity, the fill is on a horizontally layered system. The average values of γ , c and ϕ for each layer have been determined and it will be assumed c and ϕ to be laboratory values free of safety factors. The fill as well as the underlying materials is consolidated under its own weight. It is also assumed that a center of failure has been chosen for the critical circle.

As shown in the figures, layered systems are accounted for by choosing a basic material, usually the plastic layer and converting all overlying material to equivalent heights of this basic material. This could be considered as making non-homogeneous sections homogeneous. Considering this, it can be seen that the water table makes two layers of one layer of the same type of material.

Since the sand layer and layer of grey silt and clay are saturated and submerged respectively and consolidated under their own weights, any increase in load due to the addition of the fill would be absorbed by the water in the form of neutral pressure. This is the case of zero percent consolidation.

At some time later, the neutral pressure in these underlying layers is dissipated completely meaning that a 100% consolidation condition exists.

The resulting safety factors for these two conditions of consolidation have been solved for in Fig. IA.

The procedure used in determining the vectors for the normal and tangential components along the failure arch is to merely use the height of equivalent base material above a point on the arc as the weight at this point. The normal force acts on a radial line through this point on the failure arc and can be determined graphically. The tangential force is then the closing force of the polygon on the arc at the point being considered. The total normal vector is composed of both effective and neutral pressure at 0% consolidation or any consolidation less than 100%. This is shown in Fig. I.

After all these vectors have been determined at numerous points along the arc, the curves of normal pressure at 0% and 100% consolidation are plotted. The curve of tangential or shearing force is also plotted. This is shown in Fig. IA. The areas of these respective forces times the scale factor (horiz. scale x vertical scale x unit weight of base material) represents the magnitude of

overturning forces.

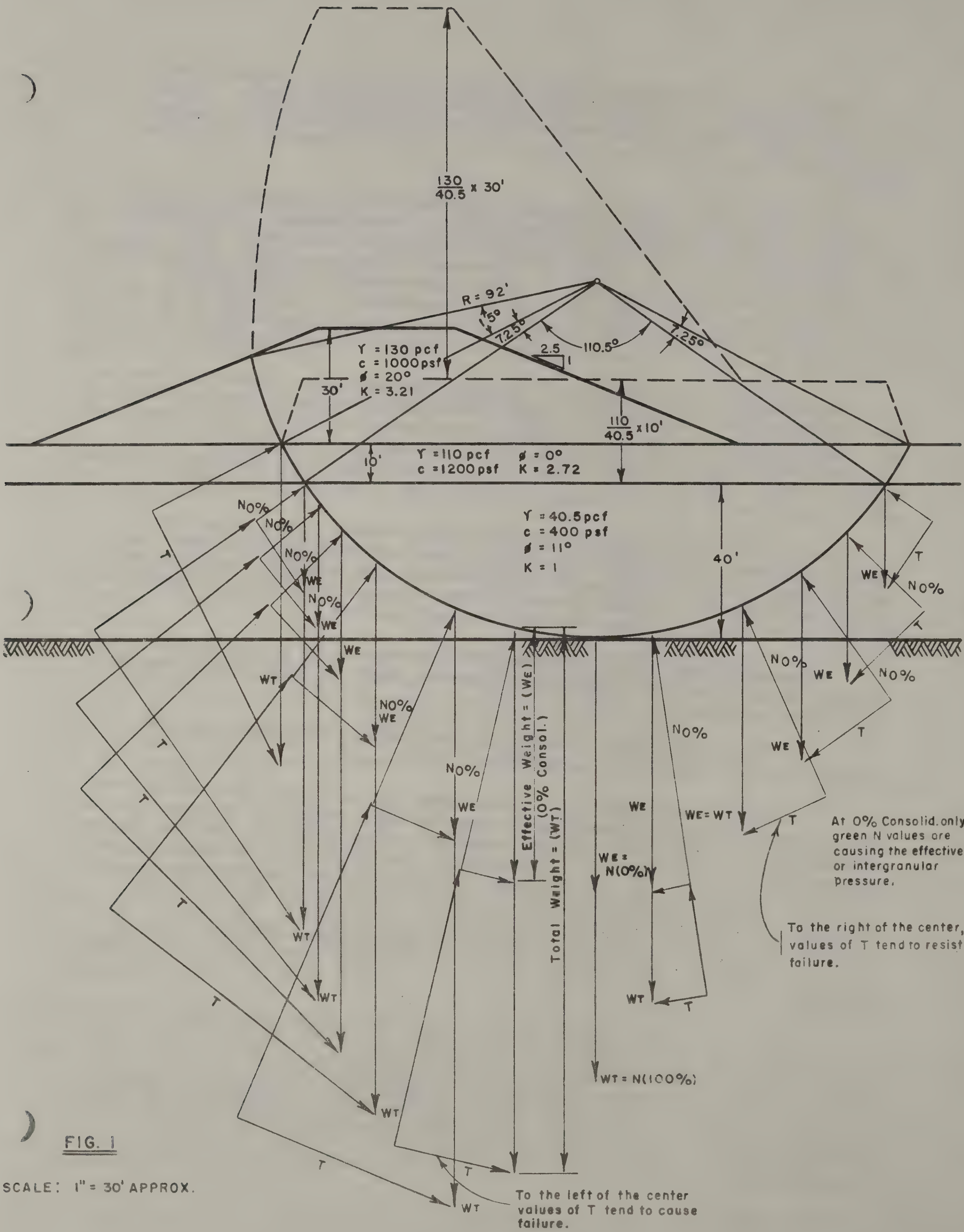


FIG. 1

SCALE: 1" = 30' APPROX.

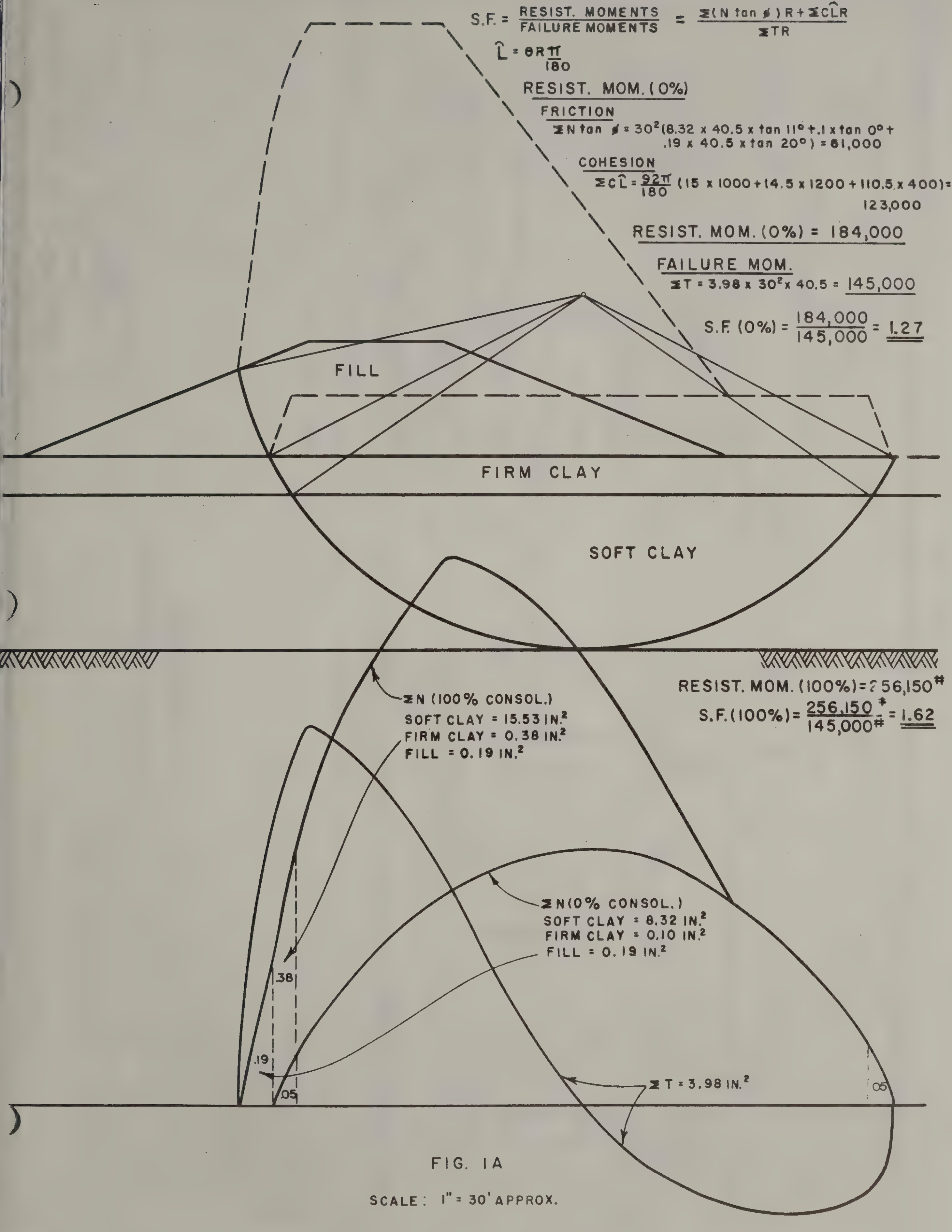
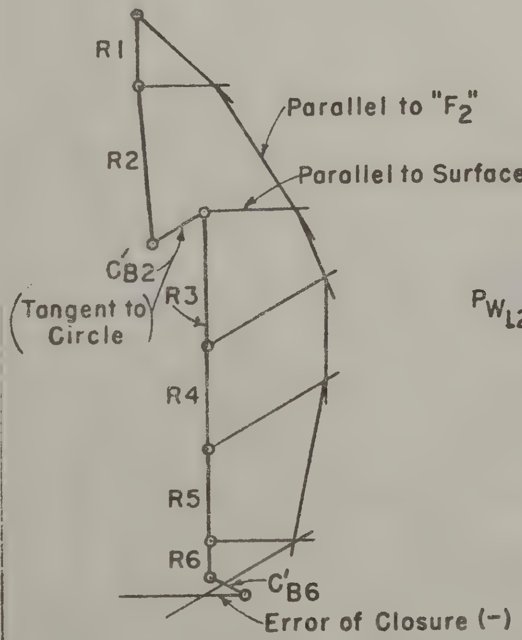
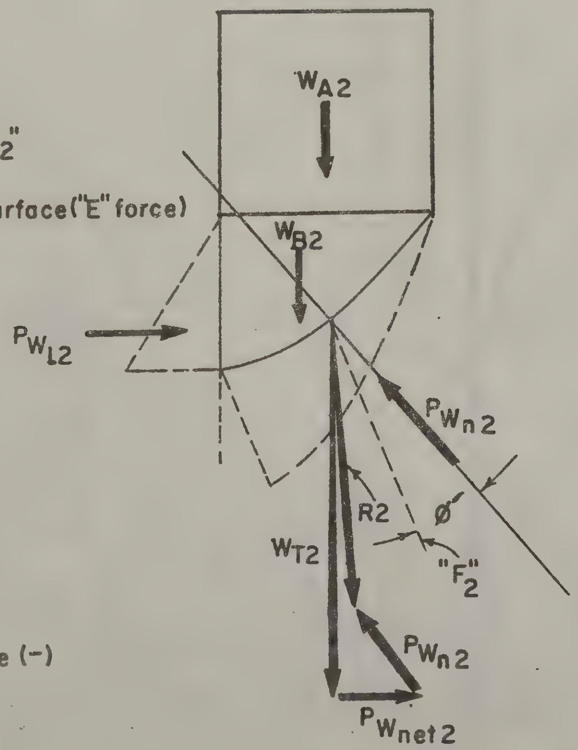


FIG. 1A

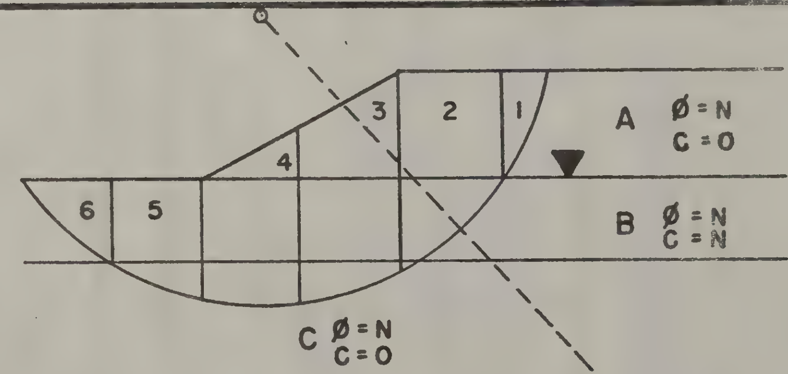
SCALE: 1" = 30' APPROX.



FORCE POLYGON



WEDGE *2



$W_{A2} = A_{A2} \times \gamma_{tA}$

$W_{B2} = A_{B2} \times \gamma_{tB}$

$W_{T2} = W_{A2} + W_{B2}$

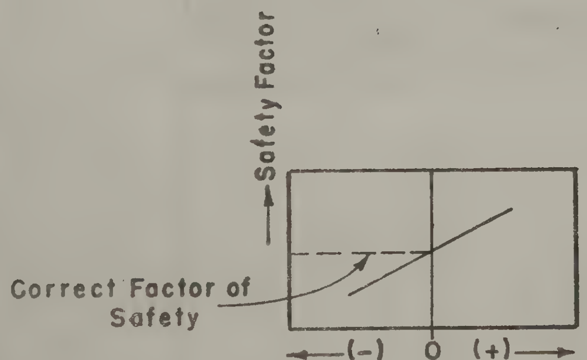
$P_{W_{L2}} = \dfrac{(h_{W_{L2}})^2}{2} \times \gamma_W$ (Water Force - Left)

$P_{W_{R2}} = \dfrac{(h_{W_{R2}})^2}{2} \times \gamma_W$ (Water Force - Right)

$P_{W_{n2}} = \dfrac{(h_{W_{L2}} + h_{W_{R2}}) \times \gamma_W \times L_{a2}}{2}$ (Water Force - Normal)

L_a = Length of Arc

$C'_{B2} = C'_B \times L_{a2}$



ERROR OF CLOSURE

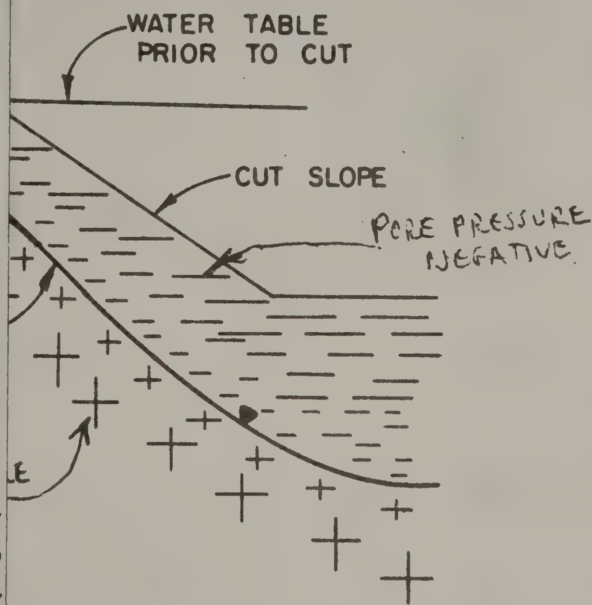
$C' = C$/Safety Factor

$\tan. \phi' = \tan. \phi$ / Safety Factor

STABILITY ANALYSIS
(Modified Swedish)
MECHANICS OF OPERATIONS

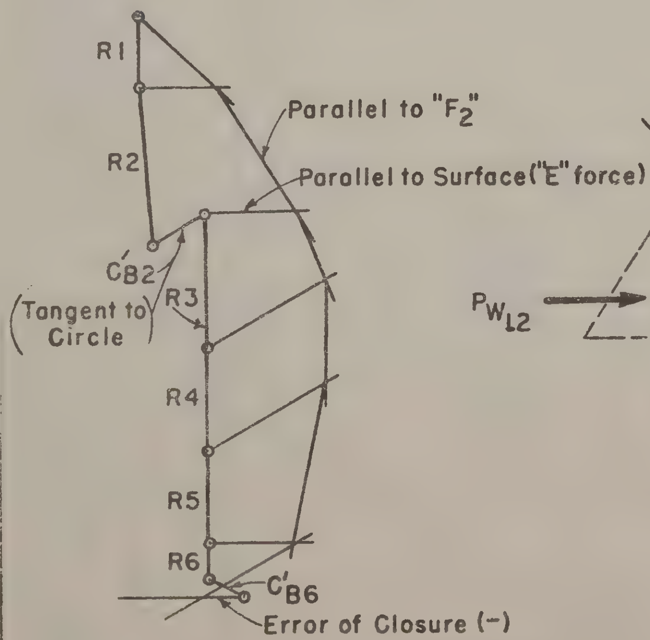
7-5

SAFETY FACTOR DEPENDS ON THE LEVEL OF EFFECTIVE STRESS WITH RESPECT TO THE DRAINED FRICTION ANGLE $= \dfrac{\text{STRESS LEVEL FOR SF=1}}{\text{ACTUAL STRESS LEVEL}}$

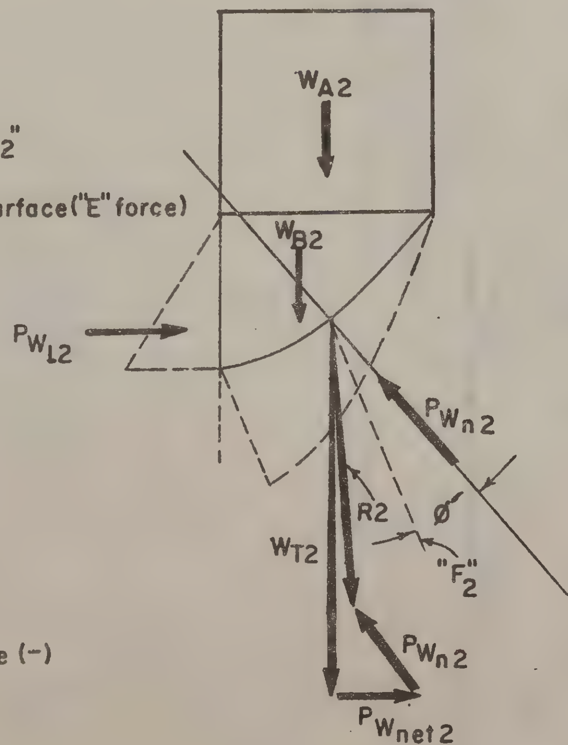


SHEARING STRESS (PSF)

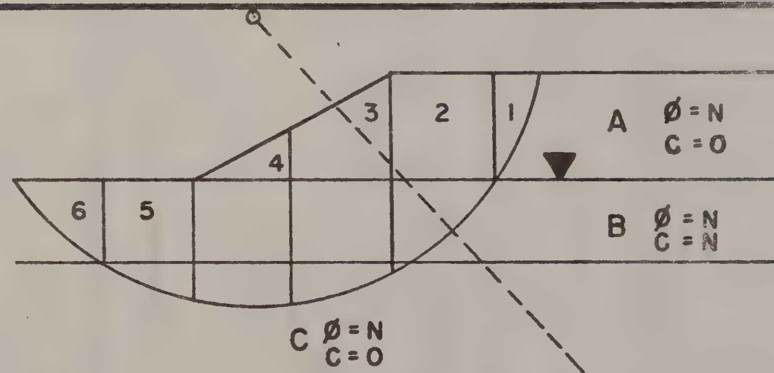
CUT SLOPE
ILURE CONCEPTS
PERMEABILITY SOILS
ASTIC CLAYS AND
LL GRADED TILLS



FORCE POLYGON



WEDGE #2



$W_{A2} = A_{A2} \times \tau_{tA}$

$W_{B2} = A_{B2} \times \tau_{tB}$

$W_{T2} = W_{A2} + W_{B2}$

$P_{W_{L2}} = \left(\frac{h_{W_{L2}}}{2}\right)^2 \times \tau_W$ (Water Force-Left)

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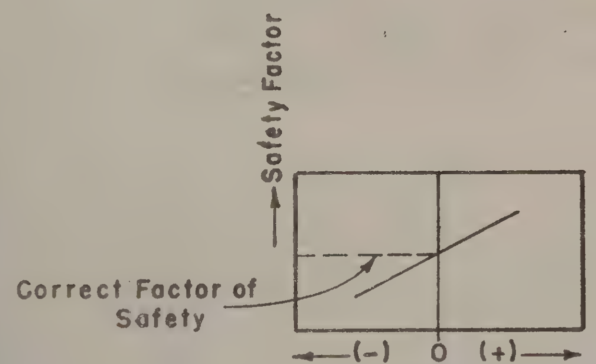
$P_{W_{n2}} = \frac{\left(h_{W_{L2}} + h_{W_{R2}}\right) \times \tau_W \times L_{a2}}{2}$ (Water Force-Normal)

L_a = Length of Arc

$C'_{B2} = C'_B \times L_{a2}$



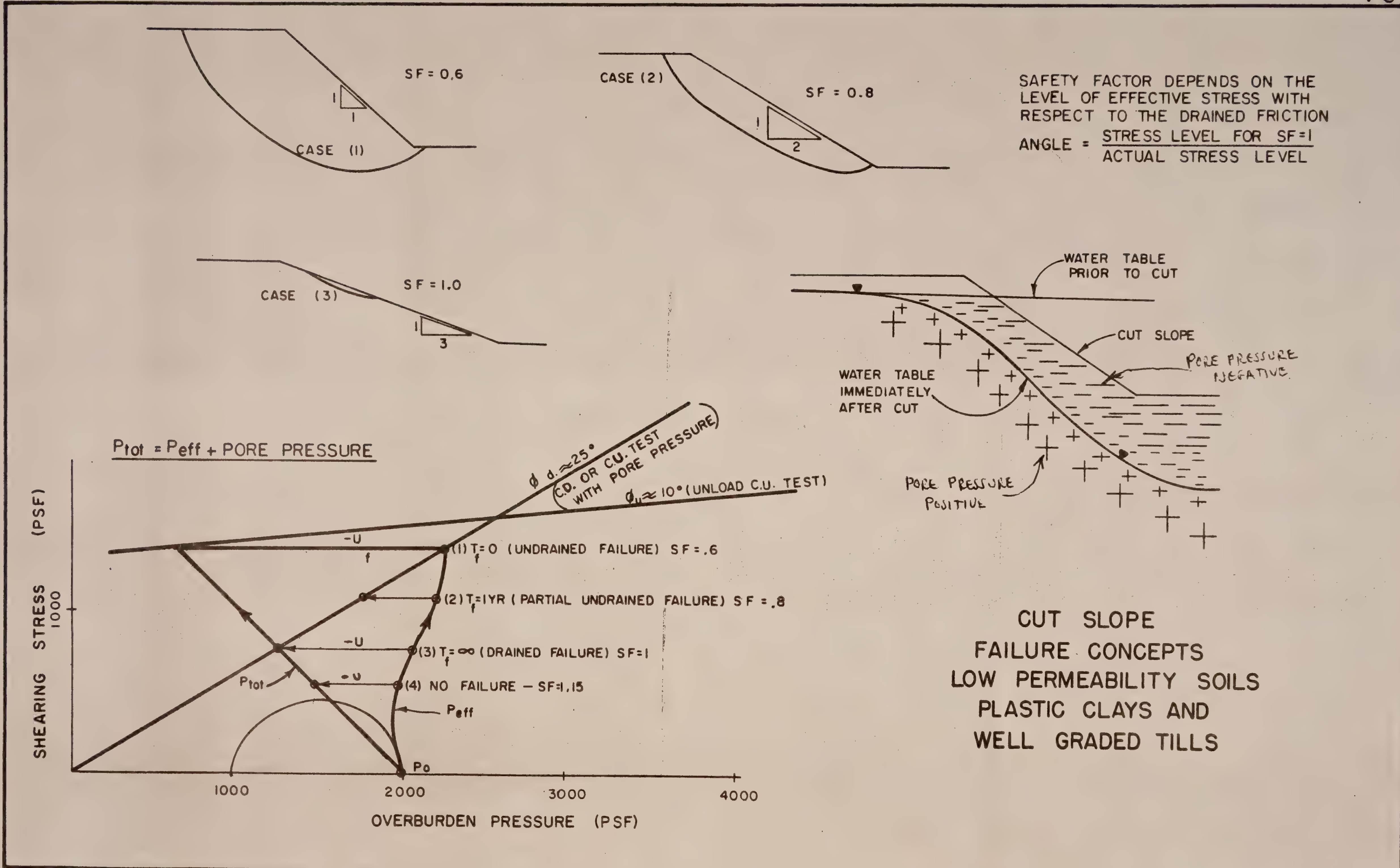
ERROR OF CLOSURE

$C' = C/\text{Safety Factor}$

$\tan.\,\text{Ø}' = \tan.\,\text{Ø}/\text{Safety Factor}$

STABILITY ANALYSIS
(Modified Swedish)
MECHANICS OF OPERATIONS

7-5



CUT SLOPE
FAILURE CONCEPTS
LOW PERMEABILITY SOILS
PLASTIC CLAYS AND
WELL GRADED TILLS

COMMENTS

DETERMINES MAGNITUDE OF SHEARING STRESSES
&
METHOD OF STABILITY ANALYSIS (ie INFINITE SLOPE OR BISHOP ANALYSIS)

- DEPTH PROGRESSED TO BELOW PROPOSED TOE DITCH. UTILIZE OBSERVATION HOLE AS PIEZOMETER IF ARTESION LAYER IS ENCOUNTERED

- SOME SAMPLE DISTURBANCE HAS LITTLE EFFECT ON RESULTS

- UTILIZE ϕ_d vs. P.I. CHART WHERE MUCH INFORMATION HAS ALREADY BEEN OBTAINED IN AREA OR FOR PRELIMINARY EVALUATIONS

DETERMINES IF RATE OF STRESS RELEASE (LOSS OF STRENGTH) WILL BE OFFSET BY DRAWDOWN(LESS SEEPAGE FORCES)

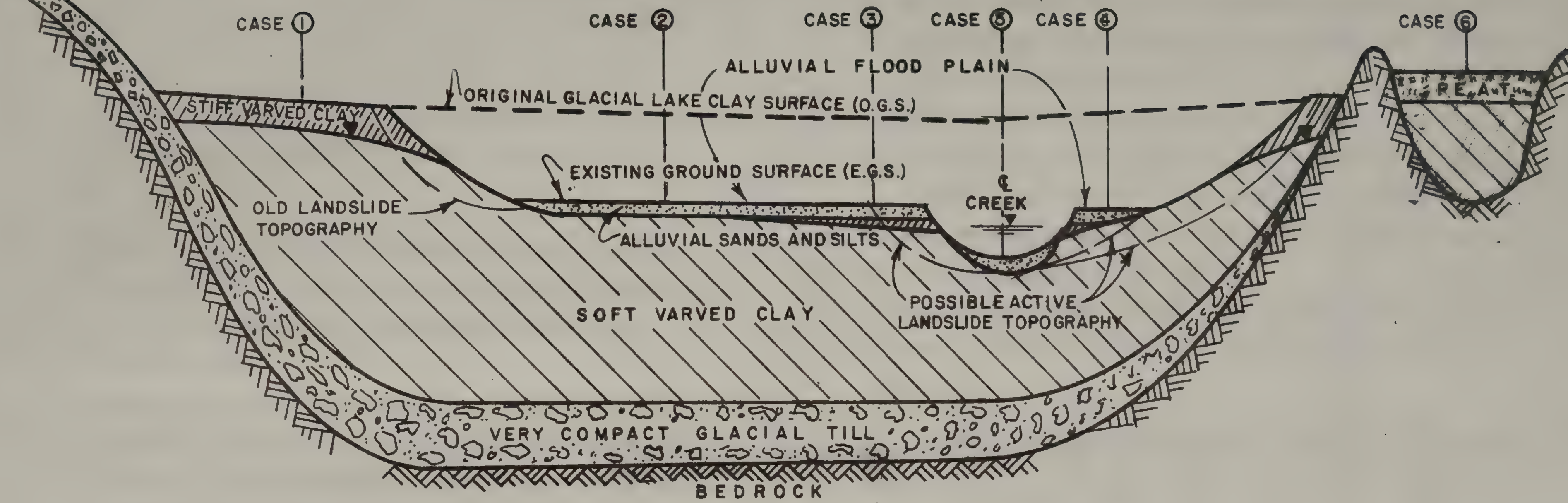
INFORMATION REQUIRED FOR CUT SLOPE DESIGN AGAINST POTENTIAL DEEP FAILURES

(PLASTIC CLAY AND TILLS - LOW PERMEABILITY SOILS)

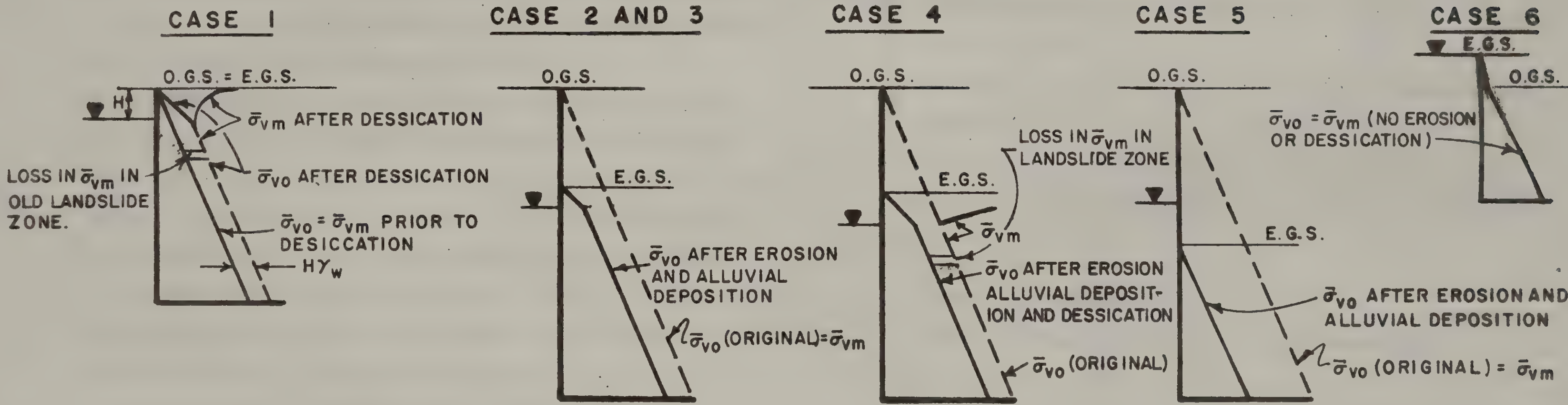
DESIGN INFORMATION	RESULTING FIELD INFORMATION	COMMENTS
DEPTH AND SLOPE OF CUT	CROSS-SECTIONS EXTENDED UPSLOPE 100-200 FEET BEYOND THE TOP OF THE PROPOSED CUTSLOPE	DETERMINES MAGNITUDE OF SHEARING STRESSES & METHOD OF STABILITY ANALYSIS (ie INFINITE SLOPE OR BISHOP ANALYSIS)
SOIL BOUNDARIES AND WATER CONDITIONS	MINIMUM OF 3 BORINGS ONE AT TOP, ONE AT BOTTOM AND ONE AT CENTERLINE OF PROPOSED CUT SLOPE TO BE CONVERTED TO WATER OBSERVATION HOLES	- DEPTH PROGRESSED TO BELOW PROPOSED TOE DITCH. UTILIZE OBSERVATION HOLE AS PIEZOMETER IF ARTESION LAYER IS ENCOUNTERED
DRAINED FRICTION ANGLE FROM C.D. TESTS OR C.U. TESTS WITH PORE PRESSURE	SHELBY TUBES IN CLAYS DENNISON SAMPLER OR BRASS LINERS IN TILLS	- SOME SAMPLE DISTURBANCE HAS LITTLE EFFECT ON RESULTS - UTILIZE ϕ_d vs. P.I. CHART WHERE MUCH INFORMATION HAS ALREADY BEEN OBTAINED IN AREA OR FOR PRELIMINARY EVALUATIONS
SOIL PERMEABILITY FROM GRADATION AND HYDROMETER ANALYSIS (BURMISTER'S CURVES IN DESIGN MANUAL OR CONSOLIDATION TESTS)	SHELBY TUBES IN CLAYS DENNISON SAMPLER OR BRASS LINERS IN TILLS	DETERMINES IF RATE OF STRESS RELEASE (LOSS OF STRENGTH) WILL BE OFFSET BY DRAWDOWN(LESS SEEPAGE FORCES)

TYPICAL SOIL PROFILE IN A VARVED CLAY DEPOSIT

WITH SUBSEQUENT ERODED VALLEY AND ALLUVIAL DEPOSITION



STRESS HISTORY



EXPLORATION AND DESIGN GUIDELINES

CASE 1

1 PROGRESS CONTINUOUS SAMPLES IN UPPER 20 TO 30 FEET TO ACCURATELY DETERMINE ELEVATION OF DESSICATED ZONE. CRITICAL STRENGTH FOR STABILITY IS USUALLY JUST BELOW DESSICATED ZONE. FIVE FOOT ERROR IN DEPTH ASSUMPTION CAN REDUCE FACTOR OF SAFETY SIGNIFICANTLY.

2 IF THE EMBANKMENT IS PLACED NEAR THE TOP OF THE CLAY SLOPE OR ON THE SLOPE, ANALYSIS MAY BE COMPLICATED BECAUSE THIS SLOPE WAS FORMED BY PREVIOUS UNDERCUTTING ACTION OF CREEK. IF THIS UNDERCUTTING WAS RAPID THERE WILL BE DEEP LANDSLIDE SCARS BENEATH THE SLOPE THAT HAVE SINCE HEALED. THE BENEFICIAL EFFECTS OF PRECOMPRESSION ON STRENGTH AND CONSOLIDATION ALONG THIS ZONE WILL HAVE BEEN ELIMINATED (ie THE SOIL WILL ACT NORMALLY CONSOLIDATED ALONG THIS ZONE.)

IN THESE ZONES, DESIGNING CUTS AND FILLS ON BULK LABORATORY STRENGTH DATA CAN BE DISASTROUS. FAILURES WILL OCCUR ON OLD SHEAR PLANES FOR SLIGHT CUTS AND FILLS. LOOK FOR USUAL EVIDENCE OF OLD LANDSLIDES (ie AERIAL PHOTOGRAPHS EXCELLENT PRELIMINARY SOURCE OF INFORMATION). FOR THESE REASONS CONTINUOUS TUBE SAMPLING TO A DEPTH OF 20 TO 30 FEET IS RECOMMENDED TO FIND OLD FAILURE SCARS ALONG THESE SLOPES.

3 THE $\bar{\sigma}_{vm}$ CURVE MAY OCCUR IN STEPS DUE TO PERIODIC DRYING (DESSICATION CURVES) OF THE GLACIAL LAKE DURING THE DEPOSITIONAL STAGE. THIS WILL COMPLICATE SETTLEMENT AND STABILITY ANALYSES.

CASE 2

DETERMINE IF FINAL PRESSURE $\bar{P}_F$ WILL BE GREATER THAN $\bar{\sigma}_{vm}$

IF $\bar{P}_F < \bar{\sigma}_{vm}$ THEN STABILITY AND SETTLEMENT PROBLEMS SHOULD BE MINOR.

CASE 3

1 SAME AS FOR CASE 2

2 ADDITIONAL FILLS OR STRUCTURES SHOULD NOT BE PLACED ON STREAM BANKS IN ZONES OF ACTIVE MOVEMENT.

3 STREAM SHOULD BE LINED WITH EROSION PROTECTION MATERIAL TO PREVENT FUTURE UNDERMINING OF PROPOSED FILLS.

CASE 4

1 SHEARING STRENGTH SHOULD BE REDUCED TO REFLECT LOWER OVERBURDEN PRESSURE IF BANKS ARE NOT ACTIVELY MOVING AND REDUCED TO THE NORMALLY CONSOLIDATED VALUE IF THEY ARE ACTIVELY MOVING (ie USED WHEN RUNNING OVERALL STABILITY ANALYSIS WITH EMBANKMENT SETBACK OUT OF FAILURE ZONE.)

2 A BORING SHOULD BE TAKEN IN THE CREEK WITH CONTINUOUS TUBE SAMPLES TO A DEPTH OF 20 TO 30 FEET.

CASE 5

FILLS AND CUTS SHOULD NOT BE PLACED IN ZONES OF ACTIVE LANDSLIDES.

CASE 6

1 CLAY IS NORMALLY CONSOLIDATED IN AREAS WHERE NO EROSION OR DESSICATION OCCURRED. USUALLY SWAMP DEPOSITS COVER THE SURFACE OF THE CLAY DEPOSIT IN THIS CASE.

2 SWAMP DEPOSITS USUALLY HAVE TO BE REMOVED AND BACKFILLED DUE TO SETTLEMENT CONSIDERATIONS. VERY OFTEN MUCH OF THE CLAY MUST ALSO BE REMOVED DUE TO THE INABILITY OF THIS MATERIAL (ie LOW STRENGTH DUE TO LOW $\bar{P}o$) TO SUPPORT ANY BACKFILL MATERIAL. THIS IS MORE OF A PROBLEM IF THE SWAMP IS DEEP.

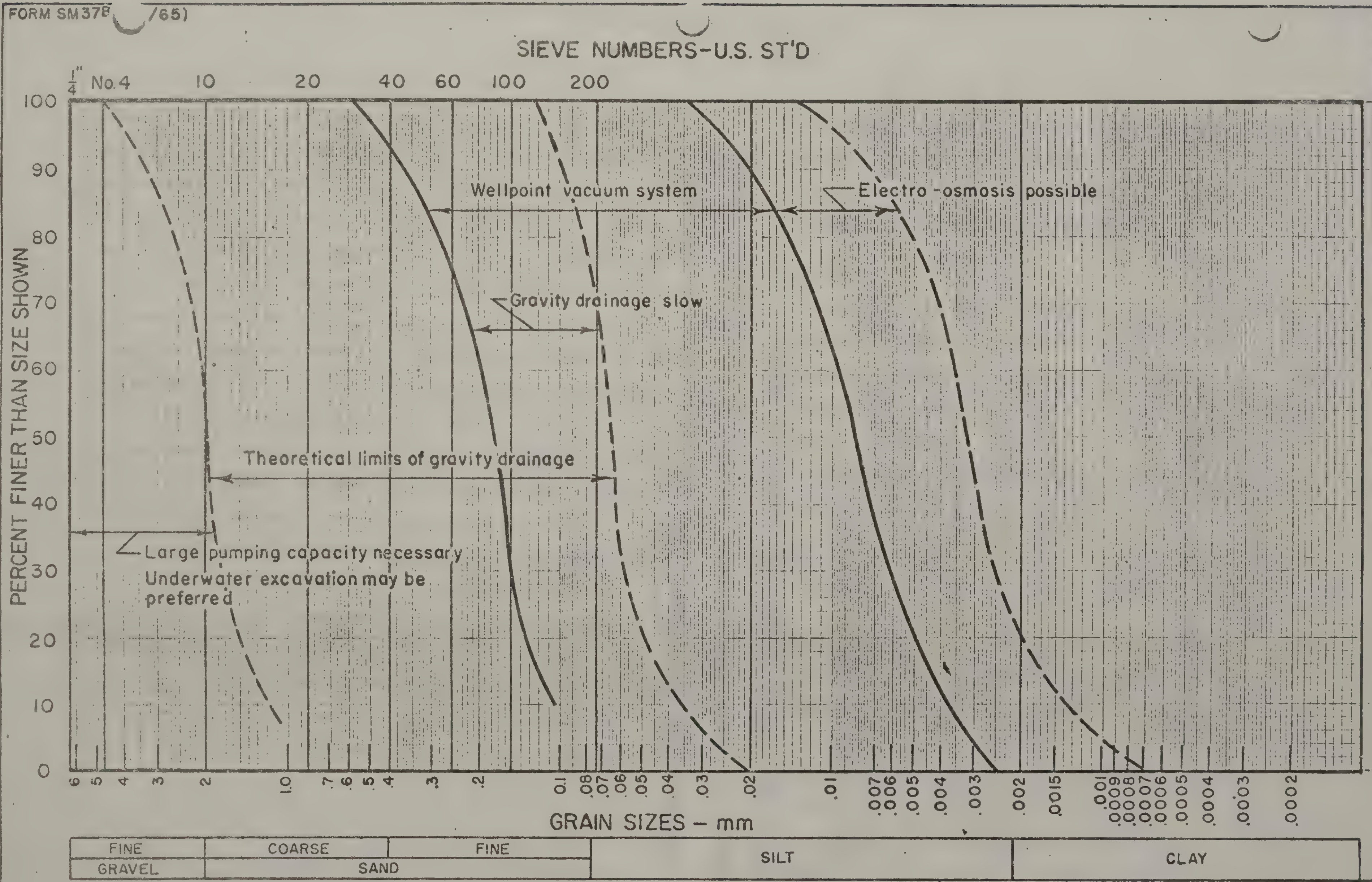
3 OBTAINING GOOD UNDISTURBED SAMPLES IS VERY DIFFICULT. THEREFORE MUST RUN SPECIAL STRENGTH TESTS TO ALLOW FOR SAMPLE DISTURBANCE AND ANISOTROPY. (ie CAU TRIAXIAL COMPRESSION TESTS CONSOLIDATED TO 4$\pm$ TIMES INSITU STRESSES AND FIELD VANE SHEAR TESTING.)

SECTION 8

SEEPAGE AND DRAINAGE ANALYSIS

PAGES

MANUALS



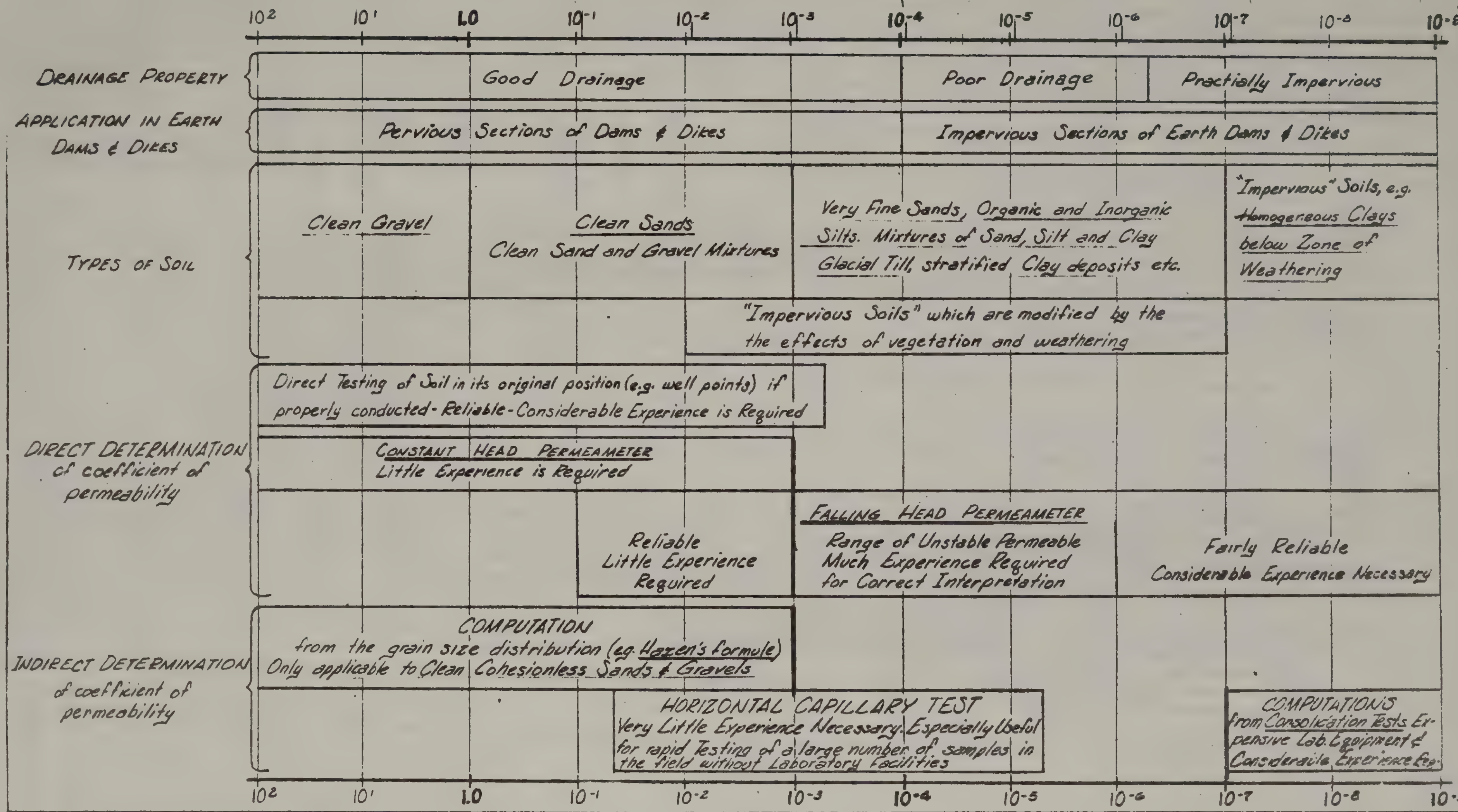
PROJECT SUITABLE SOIL DRAINAGE METHODS
SAMPLE NO. ——— DISTRICT NO. ——— COUNTY ———
STATION ——— OFFSET ——— DEPTH ———
DATE 5/25/55 DRAWN BY Wm. P. Hofmann

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS
GRAIN SIZE DISTRIBUTION CURVE

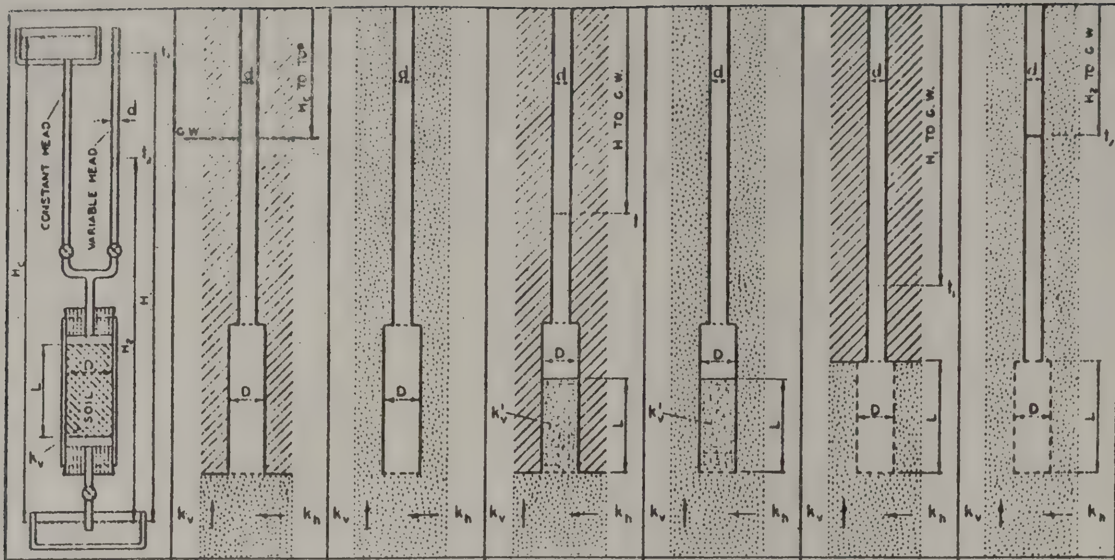
8-1

Reference: Based on experience of Moretrench Corp. from "Soil Mechanics Foundations and Earth Structures" by G.P. Tschebotarioff

COEFFICIENT OF PERMEABILITY
1ft./day = 3.52×10^{-4} cm/sec.
"k" in cm. per sec. (log scale)



NOTE: $K = 1 \times 10^{-4}$ for Fine Sand that can just about absorb heavy rains.



LABORATORY PERMEAMETER (CONSOLIDOMETER)	FLUSH BOTTOM AT IMPERVIOUS BOUNDARY	FLUSH BOTTOM IN UNIFORM SOIL	SOIL IN CASING AT IMPERVIOUS BOUNDARY	SOIL IN CASING IN UNIFORM SOIL	WELL POINT-FILTER AT IMPERVIOUS BOUNDARY	WELL POINT-FILTER IN UNIFORM SOIL
A	B	C	D	E	F	G

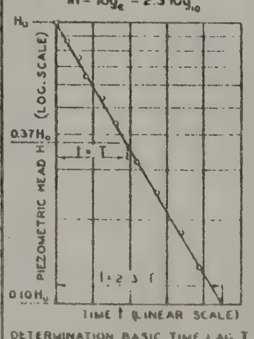
CASE	CONSTANT HEAD	VARIABLE HEAD	BASIC TIME LAG
A	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 \cdot H_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_v = \frac{L}{t_2 - t_1} \ln\frac{H_1}{H_2}$ FOR $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ FOR $d = D$
B	$k_m = \frac{q}{2 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8\,(t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ FOR $d = D$
C	$k_m = \frac{q}{2.75 \cdot D\, H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ FOR $d = D$
D	$k'_v = \frac{4 \cdot q \cdot \left[\frac{\pi}{8} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2\, H_c}$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln\frac{H_1}{H_2}$ FOR $k'_v = k_v$, $d = D$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m}\right] + L}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ FOR $k'_v = k_v$, $d = D$
E	$k'_v = \frac{4 \cdot q \cdot \left[\frac{\pi}{11} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2\, H_c}$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln\frac{H_1}{H_2}$ FOR $k'_v = k_v$, $d = D$	$k'_v = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k'_v}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot T}$ $k'_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ FOR $k'_v = k_v$, $d = D$
F	$k_h = \frac{q \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2\,\pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{2mL}{D} > 4$
G	$k_h = \frac{q\, \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2\,\pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8\, L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ FOR $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{mL}{D} > 4$

NOTATION

D = DIAM., INTAKE, SAMPLE, CM
d = DIAMETER, STANDPIPE, CM
L = LENGTH, INTAKE, SAMPLE, CM
H_c = CONSTANT PIEZ. HEAD, CM
H_1 = PIEZ. HEAD FOR $t = t_1$, CM
H_2 = PIEZ. HEAD FOR $t = t_2$, CM
q = FLOW OF WATER, CM^3/SEC.
t = TIME, SEC.
T = BASIC TIME LAG, SEC.
k'_v = VERT. PERM. CASING, CM/SEC.
k_v = VERT. PERM. GROUND, CM/SEC.
k_h = HORZ. PERM. GROUND, CM/SEC.
k_m = MEAN COEFF. PERM., CM/SEC.
m = TRANSFORMATION RATIO

$k_m = \sqrt{k_h \cdot k_v}$ $\quad m = \sqrt{k_h / k_v}$

$\ln = \log_e = 2.3 \log_{10}$



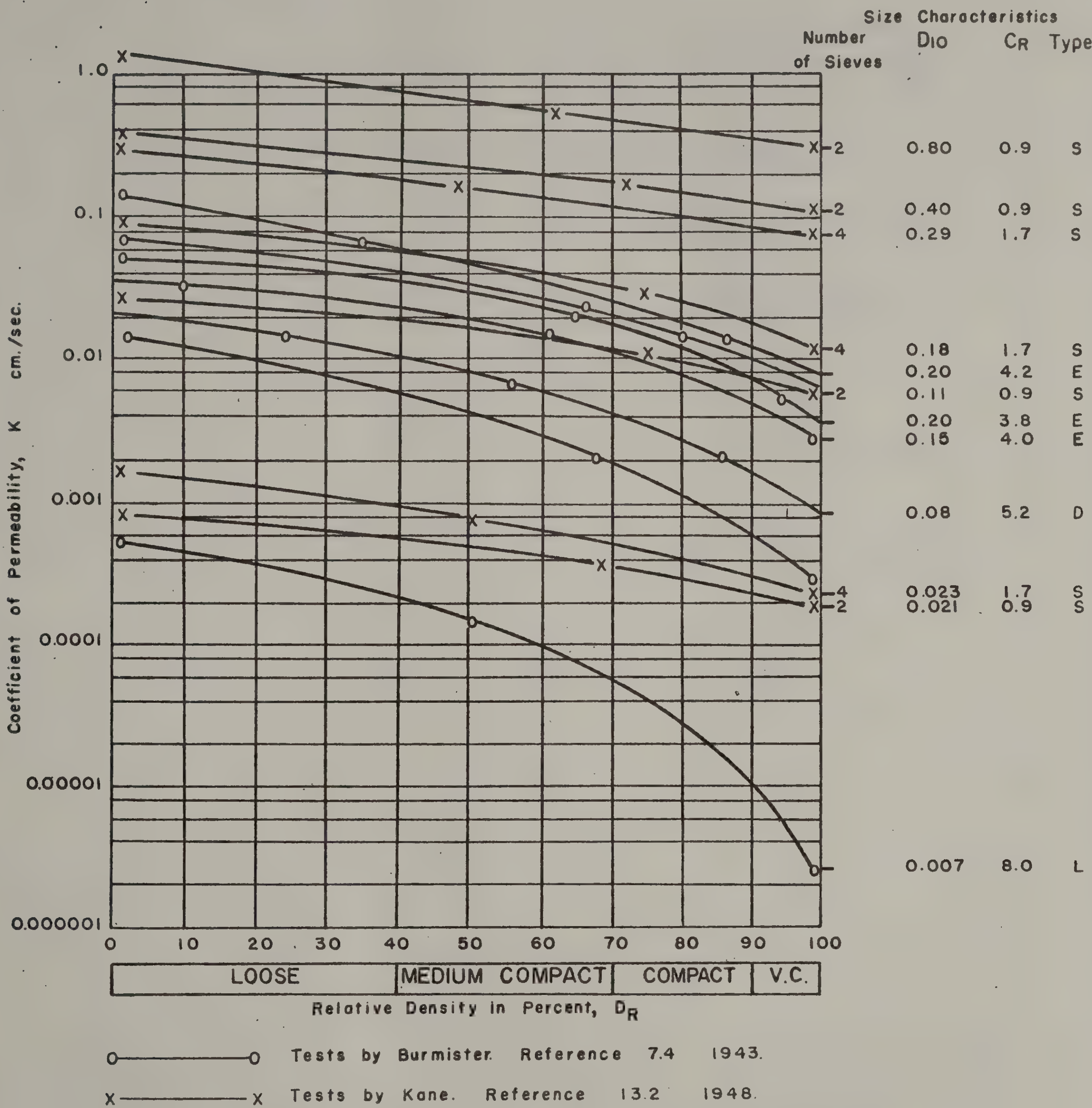
DETERMINATION BASIC TIME LAG T

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v AND k_h CONSTANT) - NO DISTURBANCE, SEGREGATION, SWELLING OR CONSOLIDATION OF SOIL - NO SEDIMENTATION OR LEAKAGE - NO AIR OR GAS IN SOIL, WELL POINT, OR PIPE - HYDRAULIC LOSSES IN PIPES, WELL POINT OR FILTER NEGLIGIBLE

Fig. 18. Formulas for determination of permeability

REF: "TIME LAG AND SOIL PERMEABILITY IN GROUND-WATER OBSERVATIONS" BULLETIN NO. 36, CORPS OF ENGINEERS WATERWAYS EXPERIMENT STATION.



Reference: Soil Mechanics Volume I
Donald M Burmister, 1960.

See: Interpretation of Grain Size Curve Characteristics, page 3-16

PERMEABILITY-RELATIVE DENSITY RELATIONSHIPS

INITIONS

CANCING PLUS AND MINUS ENCLOSED AREAS
E THE UPPER AND LOWER BRANCHES OF THE
.

UMBER OF SOIL FRACTIONS INTERCEPTED
E BETWEEN 100% AND 0% TERMINAL POINTS.
ACTIONS AT 6, 2, 0.6, .06, .02, .006,
TERS.

CURVE TO GEOLOGICAL DEPOSITS

VALLEYS, FLOOD PLAINS AND DELTAS ($+\frac{1}{4}$"
FLOW) TYPE "S" CURVE, C_R 1 TO 3, LOOSE.

ER DEPOSITED IN LAKES AND PONDS IN
C_R 1 TO 3; SEVERAL LAYERS - TYPE "S",

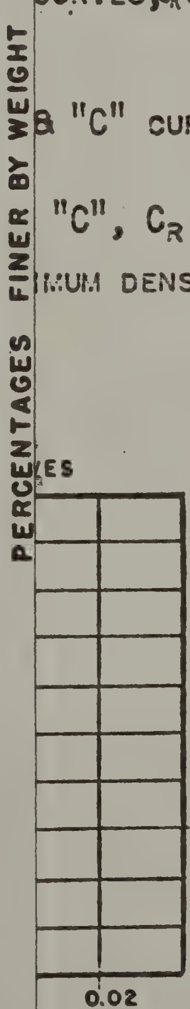
C_R 1 TO 2, LOOSE.

CURVES, C_R GREATER THAN 7-DENSE & COMPACT

& "C" CURVE, C_R GREATER THAN 5 - COMPACT,

"C", C_R 2 TO 5.

IMUM DENSITY - TYPE "C" CURVE.

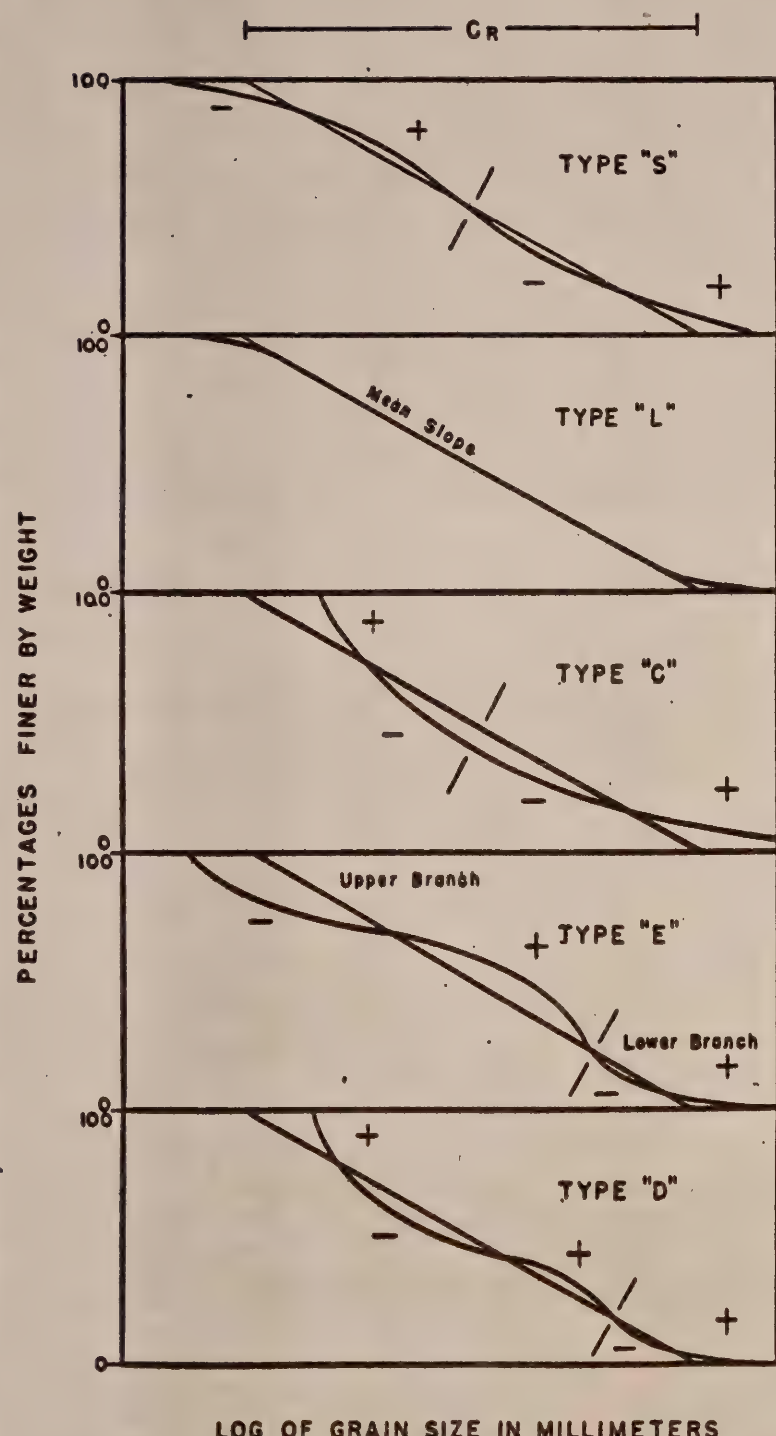


IGE OF SIZES
non plastic
Y - plastic

INTERPRETATION OF GRAIN SIZE CURVE CHARACTERISTICS

REF - SOIL MECHANICS
VOL. I BY D.M. BURMISTER
COLUMBIA UNIVERSITY

CHARACTERISTIC TYPES OF GRAIN SIZE CURVES



DESCRIPTION

S-shaped and practically symmetrically

Linear

Concave up and predominently coarse, and tapering off gradually in the fine sizes*

Convex up and predominently fine, and tapering off gradually in the coarse sizes

Irregular with a hump indicating a mixture of coarse and fine soils

DEFINITIONS

MEAN SLOPE – DETERMINED BY BALANCING PLUS AND MINUS ENCLOSED AREAS INDEPENDENTLY FOR THE UPPER AND LOWER BRANCHES OF THE GRAIN SIZE CURVE.

RANGE OF SIZE-C_R-DEFINED AS NUMBER OF SOIL FRACTIONS INTERCEPTED BY MEAN SLOPE BETWEEN 100% AND 0% TERMINAL POINTS. LIMITS OF FRACTIONS AT 6, 2, 0.6, .06, .02, .006, .002 MILLIMETERS.

RELATION OF GRAIN SIZE CURVE TO GEOLOGICAL DEPOSITS

ALLUVIAL DEPOSITS – IN RIVER VALLEYS, FLOOD PLAINS AND DELTAS (-¼" SIZE – DEPOSITED BY TURBULENT FLOW) TYPE "S" CURVE, C_R 1 TO 3, LOOSE.

ALLUVIAL DEPOSITS – QUIET WATER DEPOSITED IN LAKES AND PONDS IN LAYERS, ONE LAYER – TYPE "S" C_R 1 TO 3; SEVERAL LAYERS – TYPE "S", C_R 3 TO 5.

SAND DUNES – TYPE "S" CURVE, C_R 1 TO 2, LOOSE.

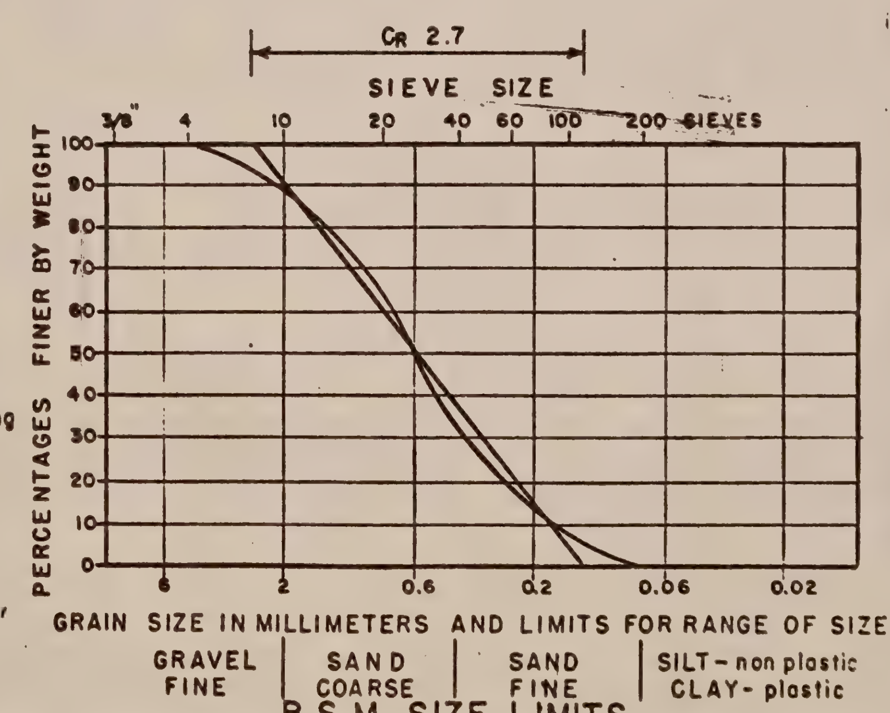
GLACIAL TILL, TYPE "D" & "L" CURVES, C_R GREATER THAN 7-DENSE & COMPACT

MODIFIED GLACIAL DEPOSITS

A. OUTWASH, TYPE "E", "D" & "C" CURVE, C_R GREATER THAN 5 – COMPACT, C_R LESS THAN 5 – LOOSE.

B. LACUSTRINE, TYPES "S" & "C", C_R 2 TO 5.

* – OPTIMUM GRADATION FOR MAXIMUM DENSITY – TYPE "C" CURVE.



INTERPRETATION OF GRAIN SIZE CURVE CHARACTERISTICS

REF – SOIL MECHANICS
VOL. I BY D.M. BURMISTER
COLUMBIA UNIVERSITY

8-6

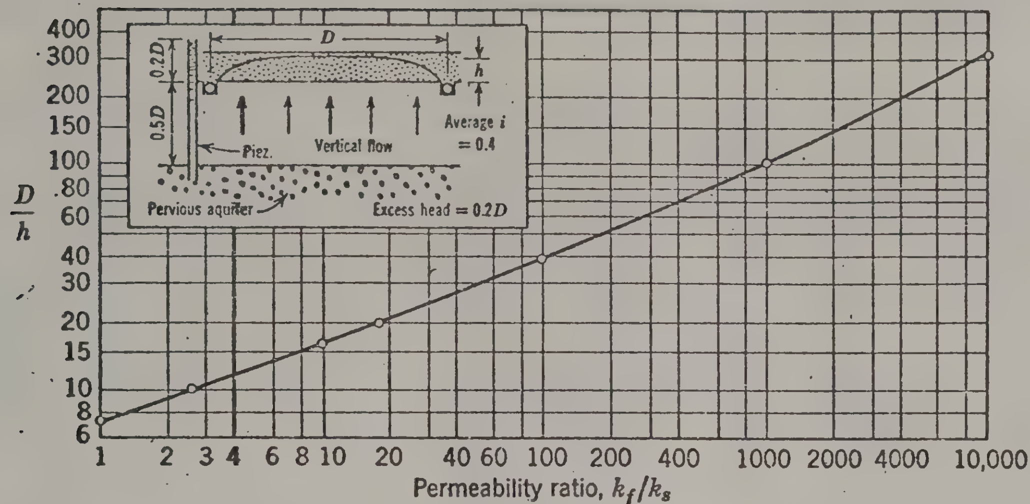


FIG. 9.16 Flow net solution for vertical seepage into horizontal blanket drains.

in the subgrade to reach the blanket drain, and in the blanket it flows horizontally to the collector drains which remove it from the roadbed (Fig. 5.11). The pervious blanket is assumed to be level, and a hydrostatic excess head of $0.2D$ is assumed in a pervious aquifer at a depth half the spacing D between pairs of collector drains. The excess head in the aquifer produces an average hydraulic gradient

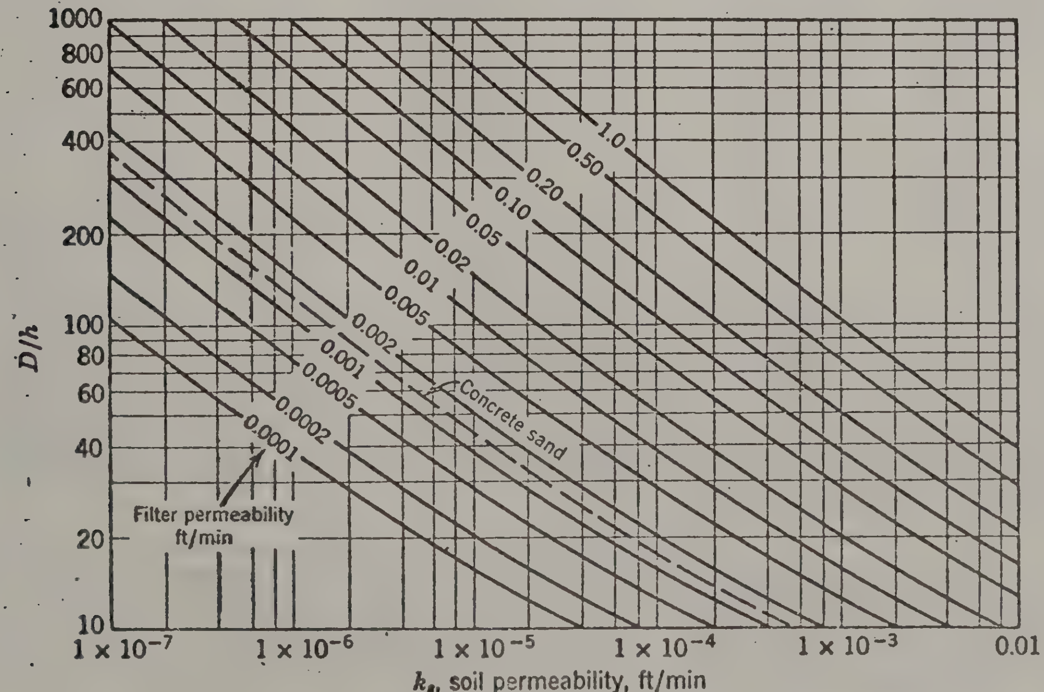
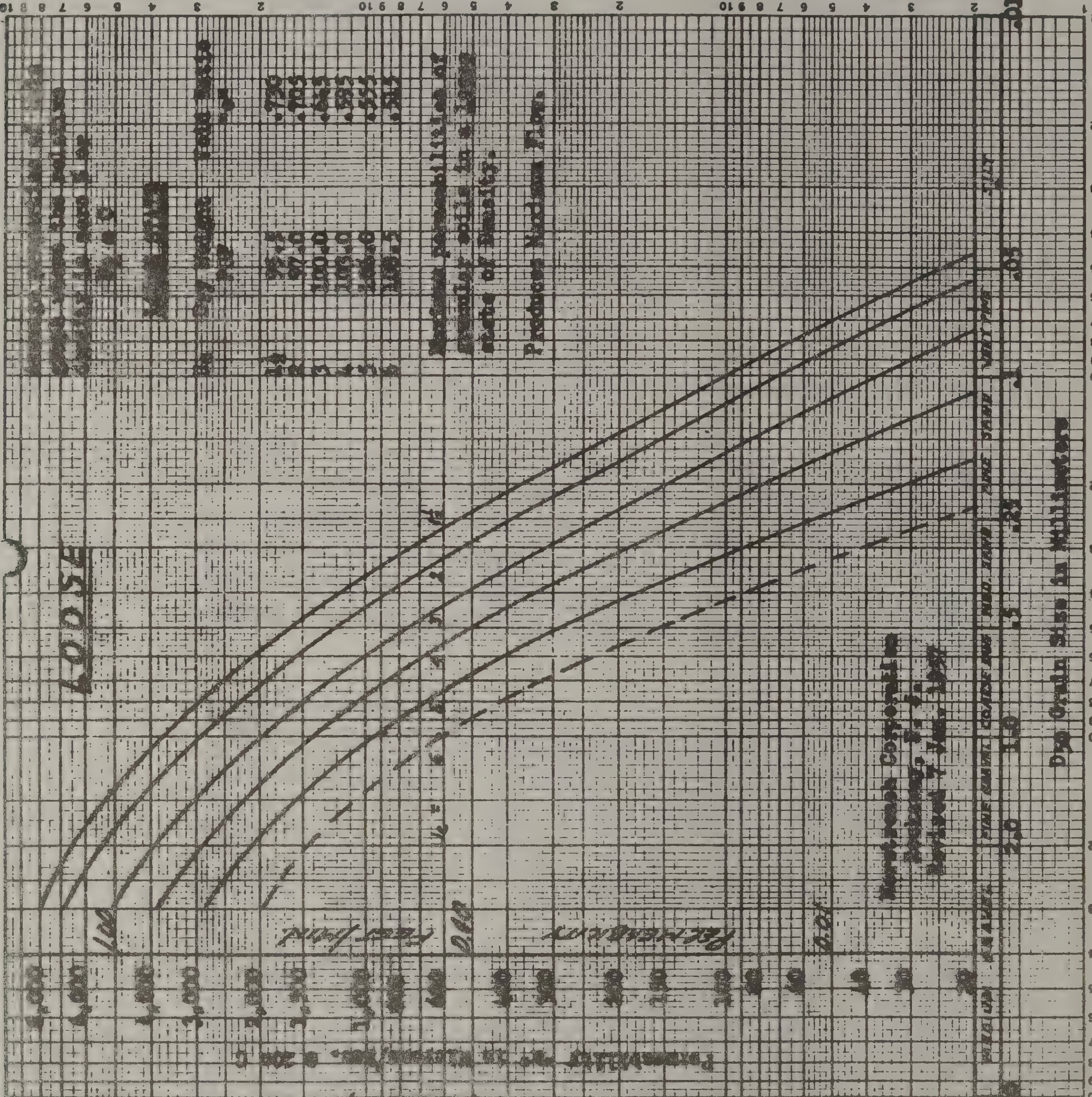


FIG. 9.17 Drain design chart developed from Fig. 9.16.

9-7

Section 430 "Conclusions From Mechanical Analysis"

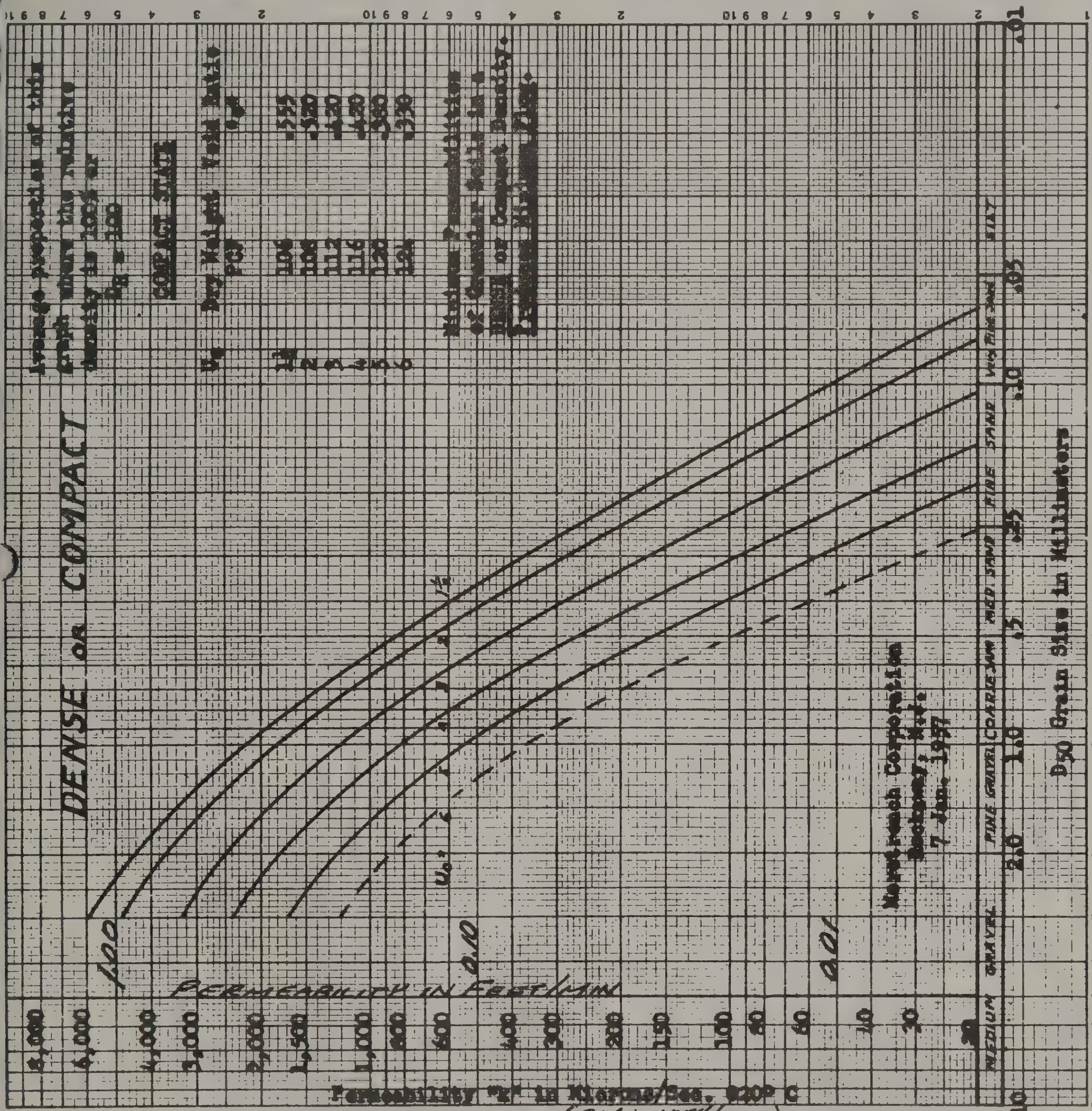
Graph showing variations in permeabilities when the sand on your job is loose. Knowing the D_{50} grain size and the U_c, find the permeability and unit dry weight from the following graph. These values are for the soil in a loose state which means maximum permeability.



From page 420.21 (opposite), you have the values of minimum permeability for soil in a dense state. Somewhere between these two values lies the permeability of the soil in its natural state of density. If you have the unit weight values or blows, you can estimate how far between these limits the true permeability is. If you don't have the actual density, assume a Relative Density of 50%.

Revised 1 October 1957

Graph showing variations in permeabilities when the sand on your job is <u>dense</u>. Knowing the D_{50} grain size and the U_c, find the permeability and unit dry weight from the above graph. These values are for the soil in dense state which means <u>minimum</u> permeability.



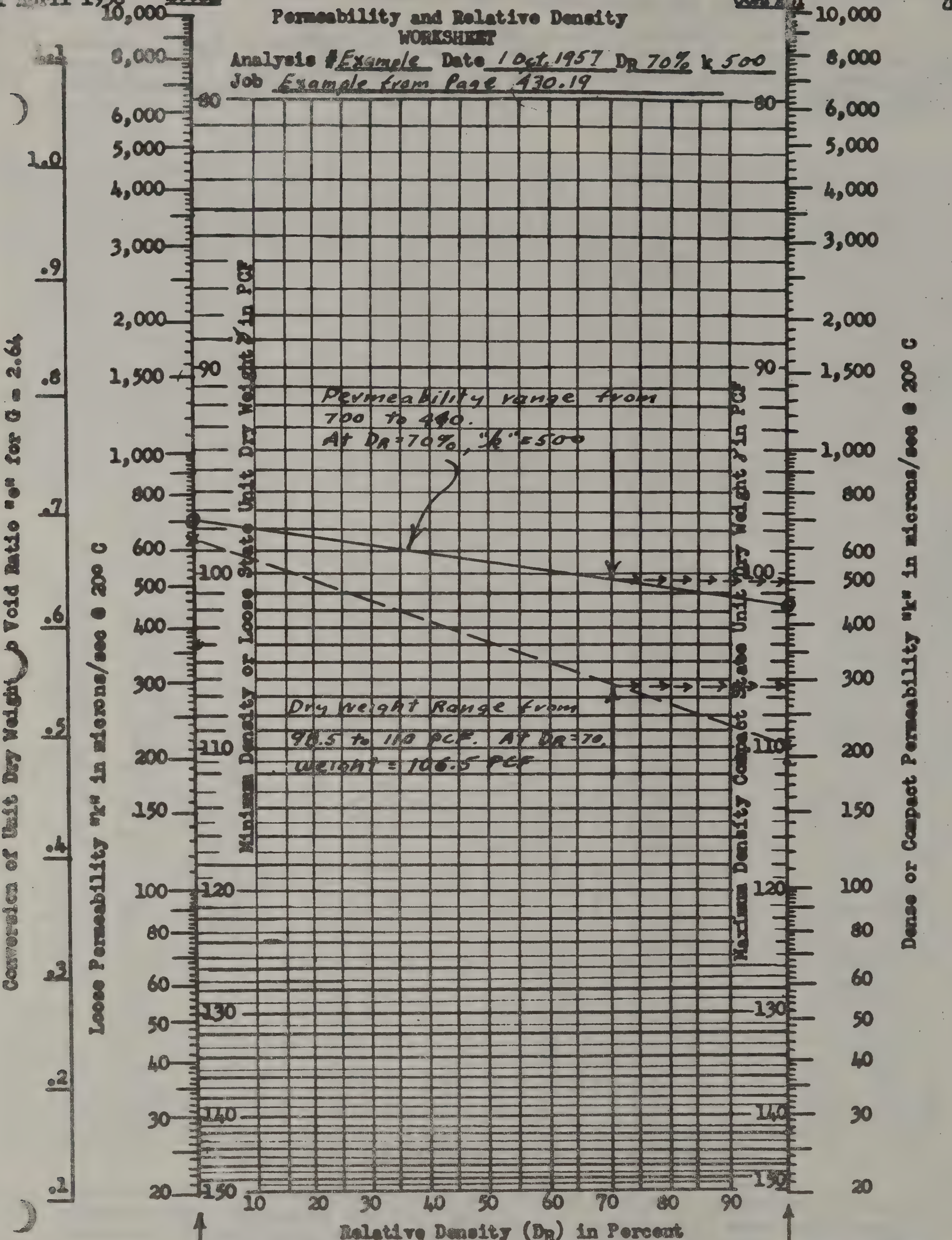
From page 420.20 (opposite), you have the values of maximum permeability for soil in a loose state. Somewhere between these two values lies the permeability of the soil in its naturl state of density. If you have the unit weight value or blows, you can estimate how far between these limits the true permeability is. If you don't have the actual density, assume a Relative Density of 50%.

Revised 1 October 1957

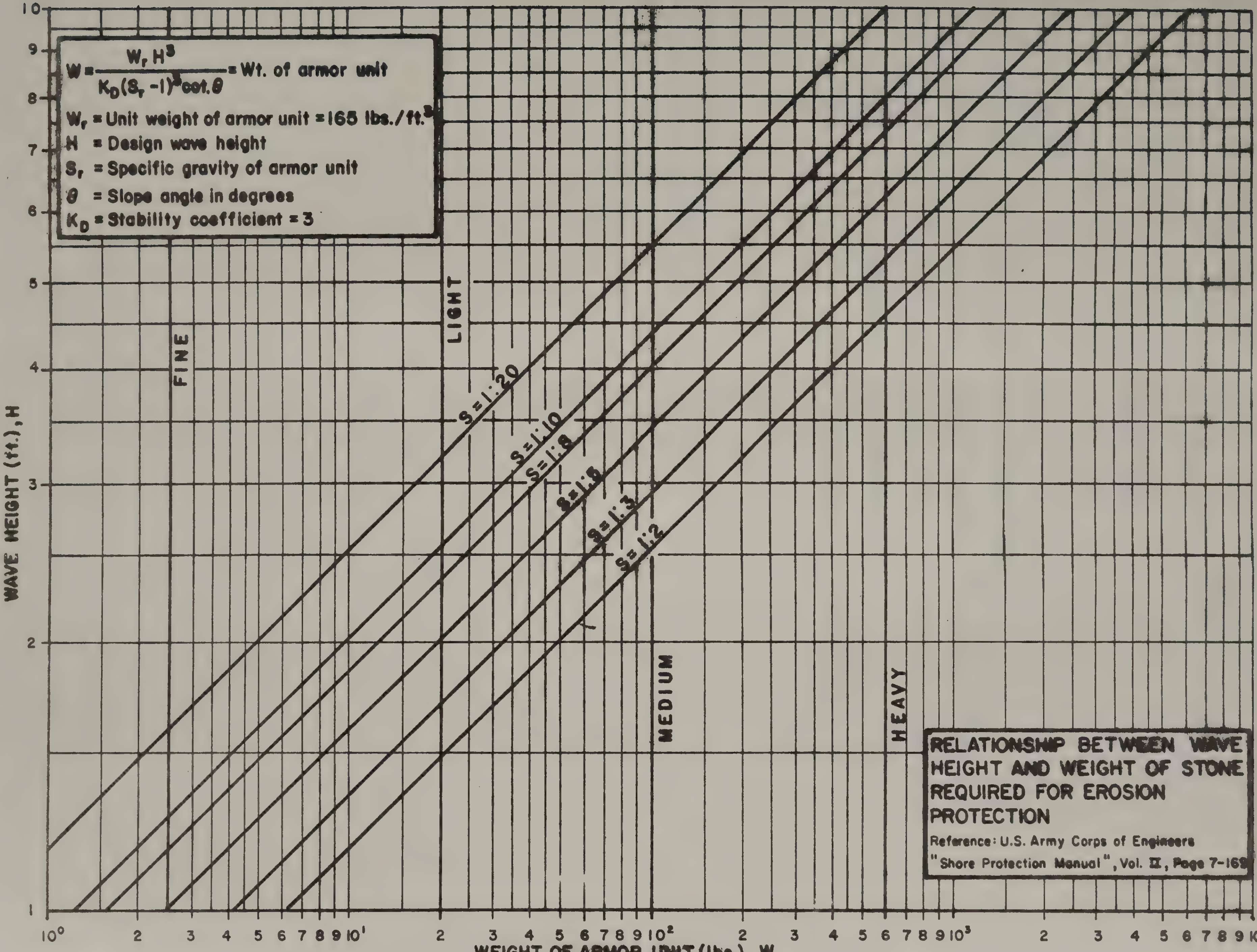
8-9

Permeability and Relative Density
WORKSHEET

Analysis # Example Date 1 Oct. 1957 D_R 70% k 500
Job Example from Page 430.19



V L	Loose	Medium (Firm)	Dense	V D	Terzaghi, Peck & Sowers
V L	Loose	Medium (Firm)	Dense	V D	Gibbs & Holtz
					13 March 1958



$W = \frac{W_r H^3}{K_D (S_r - 1)^3 \cot \theta}$ = Wt. of armor unit
W_r = Unit weight of armor unit = 165 lbs./ft.^{3}
H = Design wave height
S_r = Specific gravity of armor unit
θ = Slope angle in degrees
K_D = Stability coefficient = 3
FINE
LIGHT
MEDIUM
HEAVY
S=1:20
S=1:10
S=1:8
S=1:5
S=1:3
S=1:2
WAVE HEIGHT (ft.), H
WEIGHT OF ARMOR UNIT (lbs.), W
RELATIONSHIP BETWEEN WAVE HEIGHT AND WEIGHT OF STONE REQUIRED FOR EROSION PROTECTION
Reference: U.S. Army Corps of Engineers "Shore Protection Manual", Vol. II, Page 7-169

SECTION 9

COMPACTION

PAGES

TABLE V-1
PROCTOR TEST DATA

Test	Mold Size* In.	Vol. Cu. Ft.	Wt. of Hammer Lbs.	No. of Layers	Height of Hammer Drop-In.	No. of Blows Per Layer
Standard - Compactive Effort = 12,375 ft. lbs./cu. ft.						
Proctor	4.6 x 4.0 dia.	1/30	5-1/2	3	12	25
AASHO	4.6 x 6.0 dia.	.074	5-1/2	3	12	56
Modified - Compactive Effort = 56,250 ft. lbs/cu..ft.						
Proctor	4.6 x 4.0 dia.	1/30	10	5	18	25
AASHO	4.6 x 6.0 dia.	.074	10	5	18	56

*Molds of slightly different dimensions but of the same volumes as those listed in this table are sometimes used.

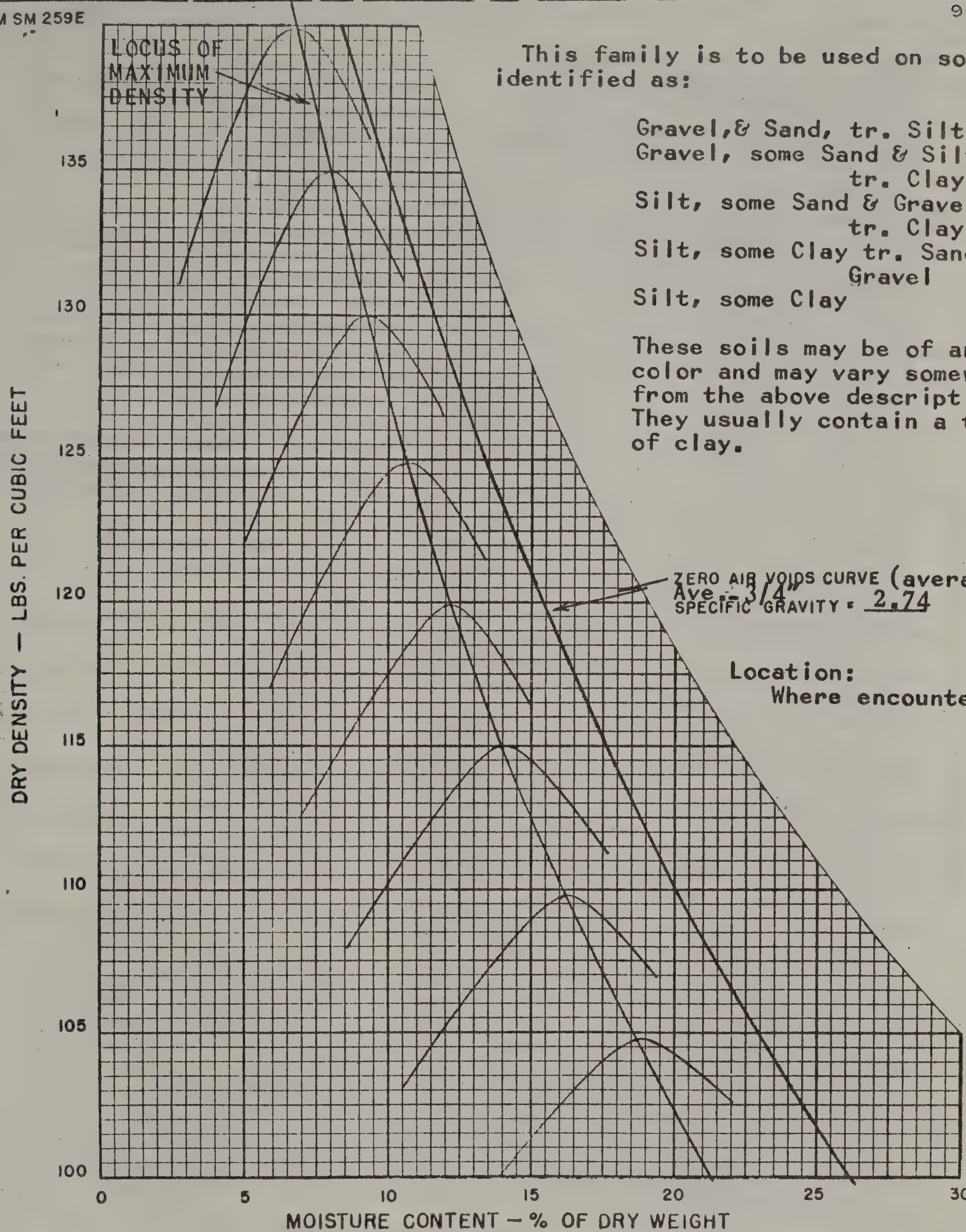
The mold of the California Bearing Ratio (CBR) Test is 7 in. x 6 in. diam., but in order to obtain approximately equal energies per volume as obtained in the small mold, the mold is only filled to a depth of 4-1/2 in. by means of a 2-1/2" spacer disk.

Top size of tested material is 3/4 inch. Correction of field density of any material containing greater than 3/4 inch stone; by formula:

$$(W.D.)\ -3/4 = \frac{W.D.\ (Total)\ X\ \%\ (-3/4)}{1.00 - \frac{(WD\ Total\ X\ \%\ (+3/4)}{(62.4\ X\ S.G.\ (+3/4)}}$$

Refer to Drawings SM 1598 A thru E

FORM SM 259E



This family is to be used on soils identified as:

Gravel, & Sand, tr. Silt
Gravel, some Sand & Silt, tr. Clay
Silt, some Sand & Gravel, tr. Clay
Silt, some Clay tr. Sand & Gravel
Silt, some Clay

These soils may be of any color and may vary somewhat from the above descriptions. They usually contain a trace of clay.

Location:
Where encountered.

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

COMPACTION CONTROL CURVES

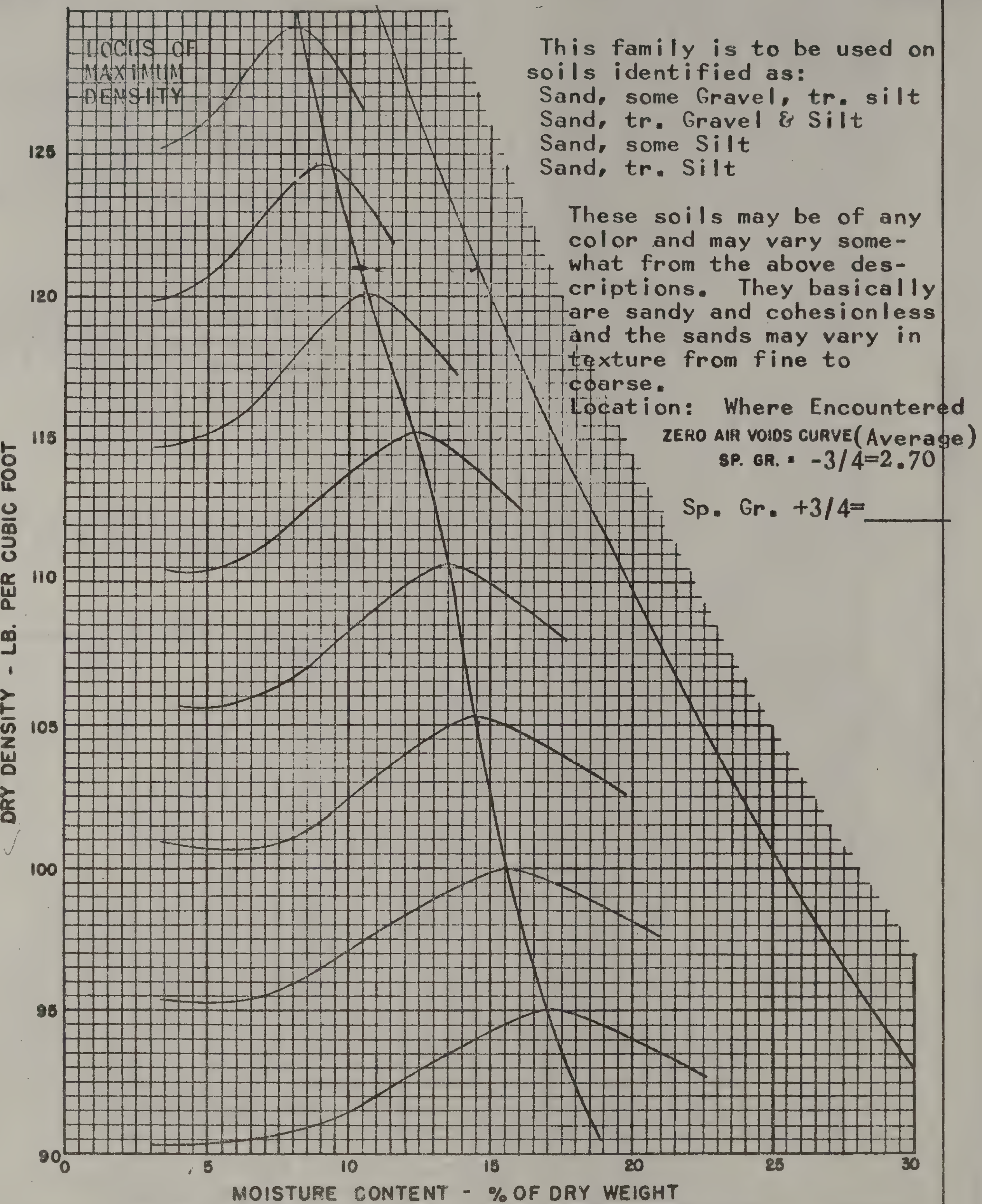
Well graded and clay

Preliminary
Sheet 1 of 2

DRAWN BY M. McC... CHECKED BY [signature]

APPROVED 1/28 1964
Wm. P. Hofmann
W. P. HOFMANN
PRINCIPAL SOILS ENGINEER

DISTRICT NO. ___
COUNTY All
DRAWING NO. ___ SM ___



Preliminary
Sheet 2 of 2

DRAWN BY: [illegible] CHECKED BY: HO

APPROVED: May 25, 1964
Wm. P. Hofmann

WM. P. HOFMANN
PRINCIPAL SOILS ENGINEER

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS

COMPACTION CONTROL CURVES

State Wide Sand Curve
Revised - Bulking

DISTRICT NO. ALL COUNTY ALL

DRAWING NO. SM

COMPACTION OF EMBANKMENTS

Purposes for the compaction of soils in an embankment:

1) Increases bearing capacity
2) Increases stability and shearing strength
3) Increases resistance to frost
4) Increases uniformity of embankment or subgrade
5) Decreases settlement
6) Decreases permeability
7) Decreases shrinkage and swell potentials

Compaction Control is the process of evaluating the compaction operations of the contractor. The embankment should be observed during compaction operations. Any areas where the construction equipment depresses or weaves the surface of the embankment are suspect. Sufficient in-place density tests should be performed in each lift to insure satisfactory compaction. Special consideratior should be accorded to areas adjacent to structures.

The steps for field compaction evaluation are:

1) In-place density in pounds per cubic feet (wet) should be determined by the sand cone method shown on page 4-6.
2) In-place density should be reduced to a -3/4 inch basis by means of the formula, on page 9-1 or by Nomographs SM 1598 A-E.
3) -3/4 inch in-place wet density should then be reduced to dry density by dividing it by 1+M.C.
4) A one-point Proctor compaction test should then be performed on the -3/4" material excavated during the in-place density test. The derived dry density and moisture contents should

then be plotted on the appropriate statewide compaction control family, pages 9-4a or 9-4b. The maximum Proctor density and optimum moisture contents are interpolated from this plot.

5) The compaction operation is evaluated by comparing, on a percent basis, the in-place density with the maximum proctor density.

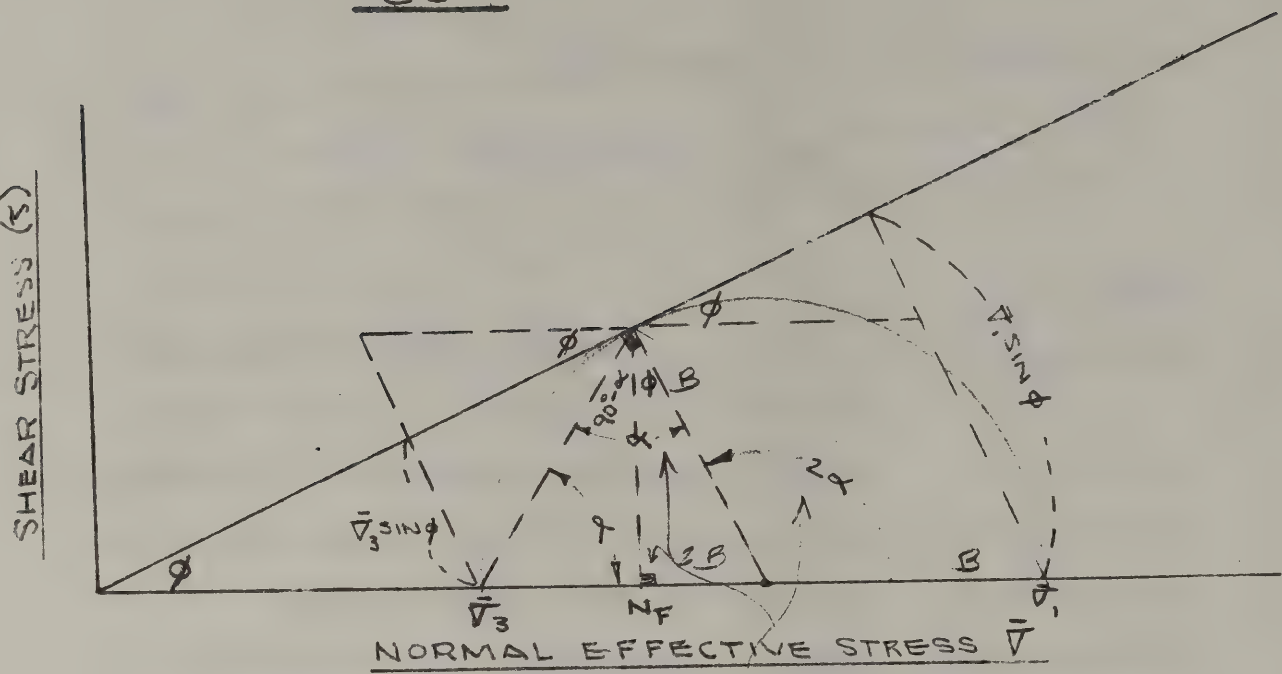
ok

SECTION 10

WALLS AND SHEETING

①

<u>C = 0</u>



(ACTIVE CASE) UNLOADING ⟵ ⟶ LOADING (PASSIVE CASE)

$\alpha = 90 - \alpha + \phi$

$2\alpha = 90° + \phi$

$2\alpha = 90 + \phi$

$$\boxed{\alpha = 45 + \phi/2}$$

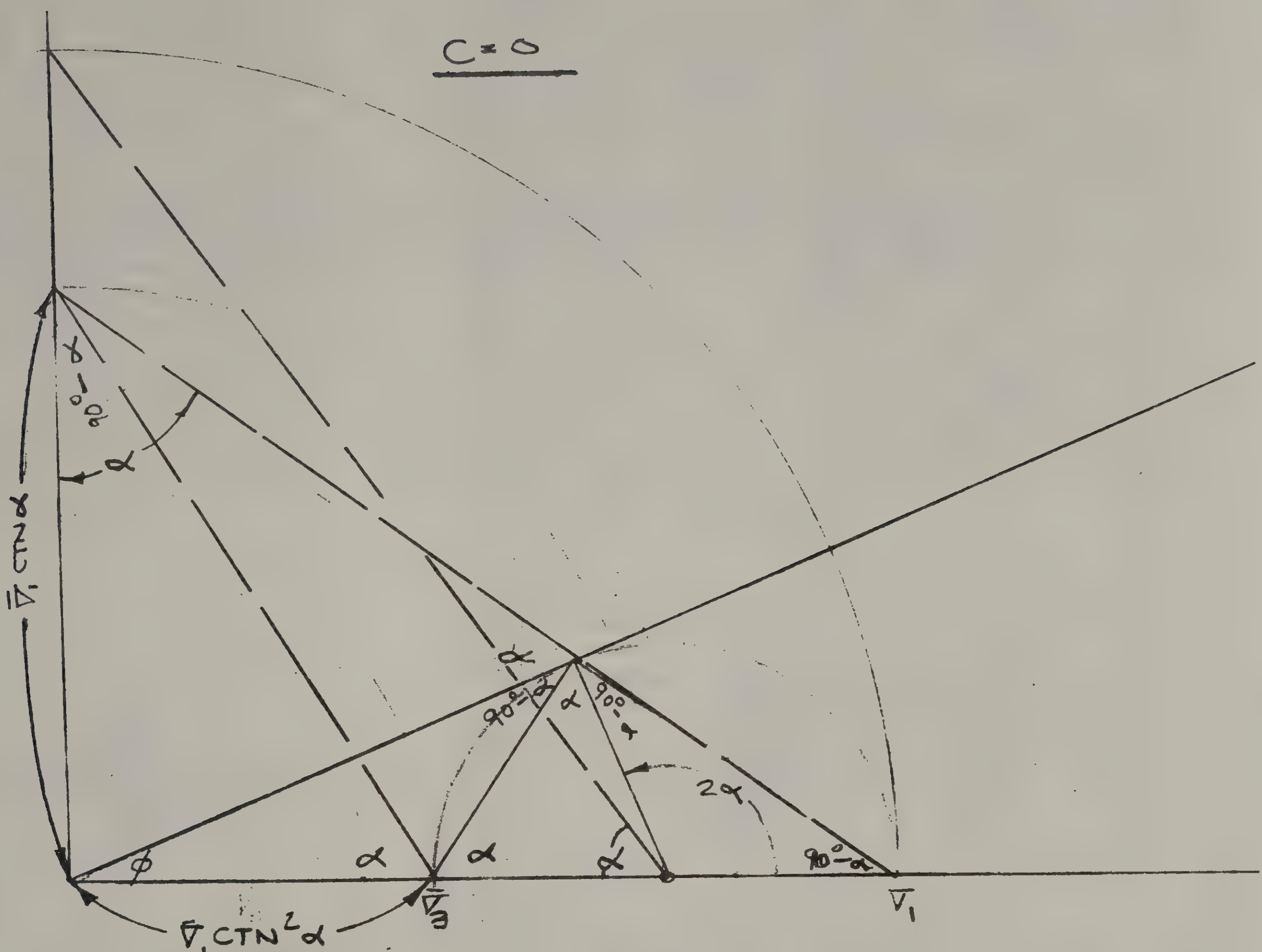
$N_F = \bar{\sigma}_3 + \bar{\sigma}_3 \sin\phi = \bar{\sigma}_1 - \bar{\sigma}_1 \sin\phi$

$= \bar{\sigma}_3(1 + \sin\phi) = \bar{\sigma}_1(1 - \sin\phi)$

$$\boxed{\begin{array}{l} K_A = \dfrac{\bar{\sigma}_3}{\bar{\sigma}_1} = \dfrac{1 - \sin\phi}{1 + \sin\phi} \\[2ex] K_P = \dfrac{\bar{\sigma}_1}{\bar{\sigma}_3} = \dfrac{1 + \sin\phi}{1 - \sin\phi} \end{array}}$$

②

C = 0



$$\bar{\sigma}_3 = \bar{\sigma}_1 \, CTN^2\alpha$$

$$K_A = \frac{\bar{\sigma}_3}{\bar{\sigma}_1} = CTN^2\alpha$$

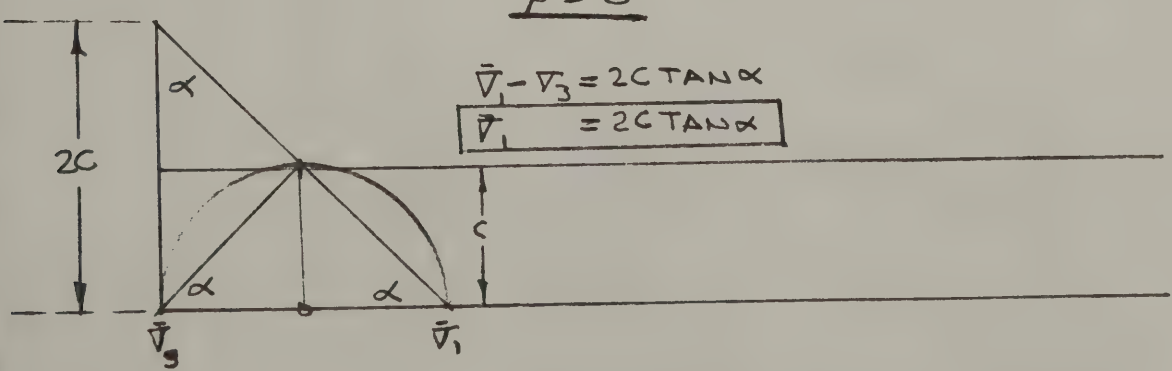
$$K_p = \frac{\bar{\sigma}_1}{\bar{\sigma}_3} = TAN^2\alpha$$

$$\boxed{\bar{\sigma}_1 = \bar{\sigma}_3 \, TAN^2\alpha}$$

$\phi = 0$

$$\bar{\sigma}_1 - \bar{\sigma}_3 = 2C\,TAN\,\alpha$$

$$\boxed{\bar{\sigma}_1 = 2C\,TAN\,\alpha}$$



ALSO

$$\frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin[2\alpha-90]}{1-\sin[2\alpha-90]} = \frac{1-\cos 2\alpha}{1+\cos 2\alpha} = \frac{1-[1-2\sin^2\alpha]}{1+[2\cos^2\alpha-1]}$$

$$= \tan^2\alpha$$

$C, \phi \neq 0$

$$\boxed{\begin{aligned} \bar{\sigma}_1 &= \bar{\sigma}_3 \tan^2\alpha + 2C\tan\alpha \\ &= \bar{\sigma}_3 \frac{1+\sin\phi}{1-\sin\phi} + 2C\tan\alpha \end{aligned}}$$

$$\boxed{K_P = \frac{\bar{\sigma}_1}{\bar{\sigma}_3} = \tan^2\alpha + \frac{2C}{\bar{\sigma}_3}\tan\alpha = \tan^2[45+\phi/2] + \frac{2C}{\bar{\sigma}_3}\tan[45+\phi/2]}$$

$$\bar{\sigma}_3 = \frac{\bar{\sigma}_1 - 2C\tan[45+\phi/2]}{\tan^2[45+\phi/2]} = \bar{\sigma}_1 \operatorname{ctn}^2[45+\phi/2] - 2C\operatorname{ctn}[45+\phi/2]$$

$$= \bar{\sigma}_1 \tan^2[45-\phi/2] - 2C\tan[45-\phi/2]$$

$$\boxed{K_A = \frac{\bar{\sigma}_3}{\bar{\sigma}_1} = \tan^2[45-\phi/2] - \frac{2C}{\bar{\sigma}_1}\tan[45-\phi/2]}$$

basements in order to assess the hindrance or help that they may provide.

b. Subsurface Investigation – Borings, water level determinations, sampling, and soil testing are required in order to determine the parameters from which earth pressures are calculated.

c. Earth Pressure – The total force per unit of length of excavation can be calculated for a given excavation depth by means of the Rankine theory of active earth pressure. Actually, it is believed that the outward movement of the sides of the excavations are generally sufficient to create an active state of stress. The Rankine theory gives a horizontal force. However, if the supports are inclined and do not yield vertically, Fig. 1(a), the total force is slightly modified on account of the wall friction. This force may be calculated by the Caquot-Kerisel theory (Caquot et al, 1948).

The sum of the actual strut loads, $P_1 + P_2 + P_3$ in Fig. 1(a), is roughly equal to the total force P_A given by the Rankine theory; however, the distribution of the total force amongst P_1, P_2, and P_3 depends to a great extent upon the construction procedures. Therefore, the struts should be designed for loads Q_1, Q_2, and Q_3, the sum of which is often 20% to 50% greater than P. Fig. 2 proposes rules which are adapted from the rules suggested by K. S. Flaate (Flaate, 1966).

The design of the wales and walls of the excavation is less critical than the design of the struts. Whereas, the latter may fail by buckling with little warning, the former are stressed in bending; therefore, overstressing increases deflection which reduces bending stresses because of arching. Fig. 1(b) shows assumptions which are often made when designing steel struts and wales. Because construction materials commercially available are adequate for the support of excavations with depth smaller than about 100 ft, the design of the structural supports rarely is difficult.

d. Heave and Blowout – The possibility of heave, or bottom failure of the excavation, should be investigated. Fig. 3 contains a sample of the calculations which were made for an actual design. The Terzaghi and Peck method (Terzaghi et al, 1967) or the method described by Bjerrum and Eide (Bjerrum et al, 1956) are recommended for use.

In clay, a plastic zone forms below the bottom of the excavation. The depth of the plastic zone depends upon the height, H, and the width, B, of the excavation. The magnitude of the heave is related to the stability number $N = \gamma H/c$. When N is

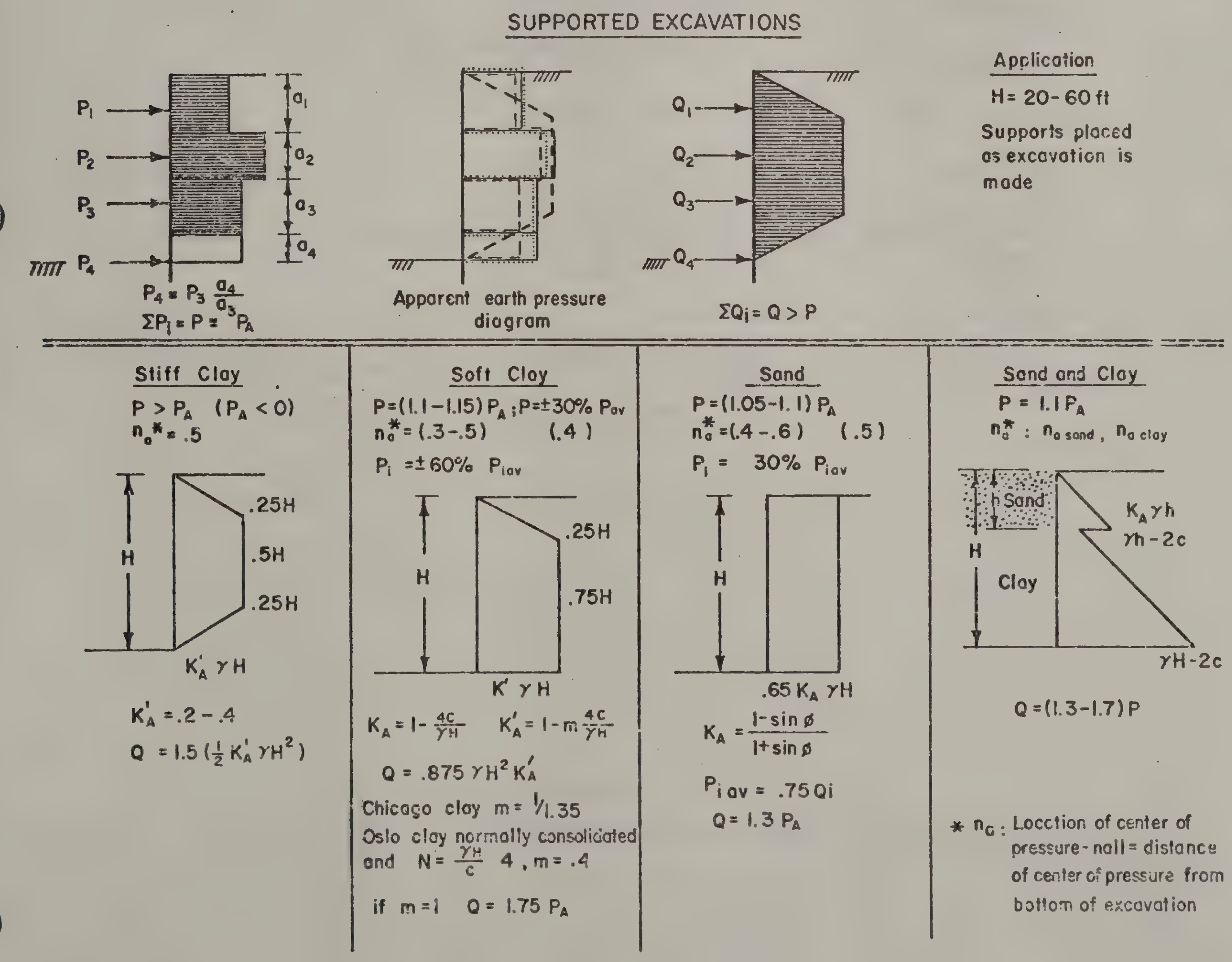
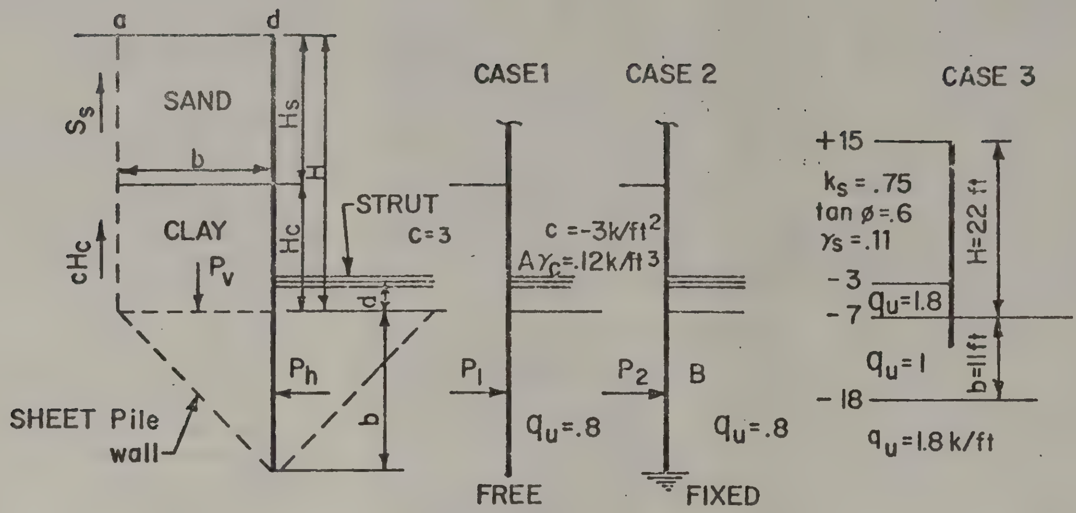


FIGURE 2 SUPPORTED EXCAVATIONS



$S_s = 1/2\gamma_s H_s^2$ ks tan o $= .0247\ H_s^2$ k/ft³

FREE END	FIXED END
$P_v\ \gamma H - \frac{1}{b}(S_s + cH_c)$	$P_v = \gamma H - -(2 S_s + 2_c H_c + \frac{q_u}{2} b)$
$Ph = P_v - \pi \frac{qu}{2}$	$P_h = P_v - \pi \frac{qu}{2}$
$P_1 = P_h - qu$	$P_2 = P_h - q_u$
$Mt\,(A) = P_1 \frac{b^2}{2}(1 + 2\frac{d}{b})$	$M_t\,(B) = P_2 \frac{b^2}{8}(1 + 2\frac{d}{b})$
$f_s = \frac{Mt\,(A)}{s}$	$f_s = \frac{Mt\,(B)}{s}$

ASSUME MZ 32 → S = 38 in³/ft /in

CASE	H ft	H_s ft	b ft	d ft	TYPE OF SPT	S_s k/ft lin	c Hc k/ft lin	γH k/ft²	P_v	P_1	MAX MOMENT kx in/ft lin	f_s k/in²
1	30	15	20	5	FREE	5.56	4.5	3.45	2.95	0.89	2670	70
2	11	11	30	11	FIXED	11	11	11	2.38	0.32	508	13
3	22	18	11	N.A.	FREE	8.03	3.6	2.46	1.40	0	0	0

FIGURE 3 HEAVE AND DEPTH EMBEDMENT

greater than 4, heave should be expected. Large heave is likely to occur when N = 6 to 8.

Blowouts through vertical circular openings between soldier beams or lagging boards should be expected when N = γ H/c = 6 to 8 (Broms, 1967).

e. Estimate of Movement – The magnitude of the lateral movement and settlement of the side of an excavation and adjacent properties cannot be calculated.

In clay, significant movements usually take place when N = γ H/c is greater than 3 or 4. When N is between 5 and 10, settlements as great as one or two percent of the excavation depth cannot be avoided. Settlements are due to two causes: (1) deflection of the wall of the excavation and heave; and (2) consolidation.

f. Type of Supports and Construction Sequence – The choice of the type of supports and the planning of the construction sequence are generally considered to be the prerogative of the contractor. This view is sound but should not prevent the engineer from preparing a design memorandum in which the relative merits of several procedures are outlined.

Because the magnitude of the movements of the adjacent properties depend upon the length of time during which the excavation remains open, the work schedule should be arranged so that the length of time the excavation remains open in the vicinity of sensitive structures is minimum. Because experience is always acquired during the first stages of the work, it is recommended that construction be started in the less critical portions of the excavation to obtain the maximum profit from the observations made at the beginning of the project.

Table I gives a list of usual construction procedures.

The following is a list of statements which may be used as guides when the type of support and construction sequence are being determined.

— Staged construction leads to less movement of the neighboring properties and is not necessarily more expensive than excavating the entire site before starting construction of the permanent structure.

— Walls consisting of steel sheet piles or reinforced concrete subpiers which are later used as part of the permanent wall are very satisfactory; this also applies to a cast-in-place reinforced concrete wall built in a slurry trench.

— Soldier beams, such as H-piles, or in some cases, cast-in-place reinforced concrete piles, are very economical. They are sometimes necessary when sheet piles cannot be driven. Timber lagging is installed between the soldiers as the excavation proceeds. To permit arching and reduce settlement, lagging should not be placed too far behind the soldier beams; see Fig. 4.

— Some settlement of the adjacent ground cannot be avoided because the sides of the excavation have the tendency to move below excavation level; that is, before the supports can be installed; see Fig. 5.

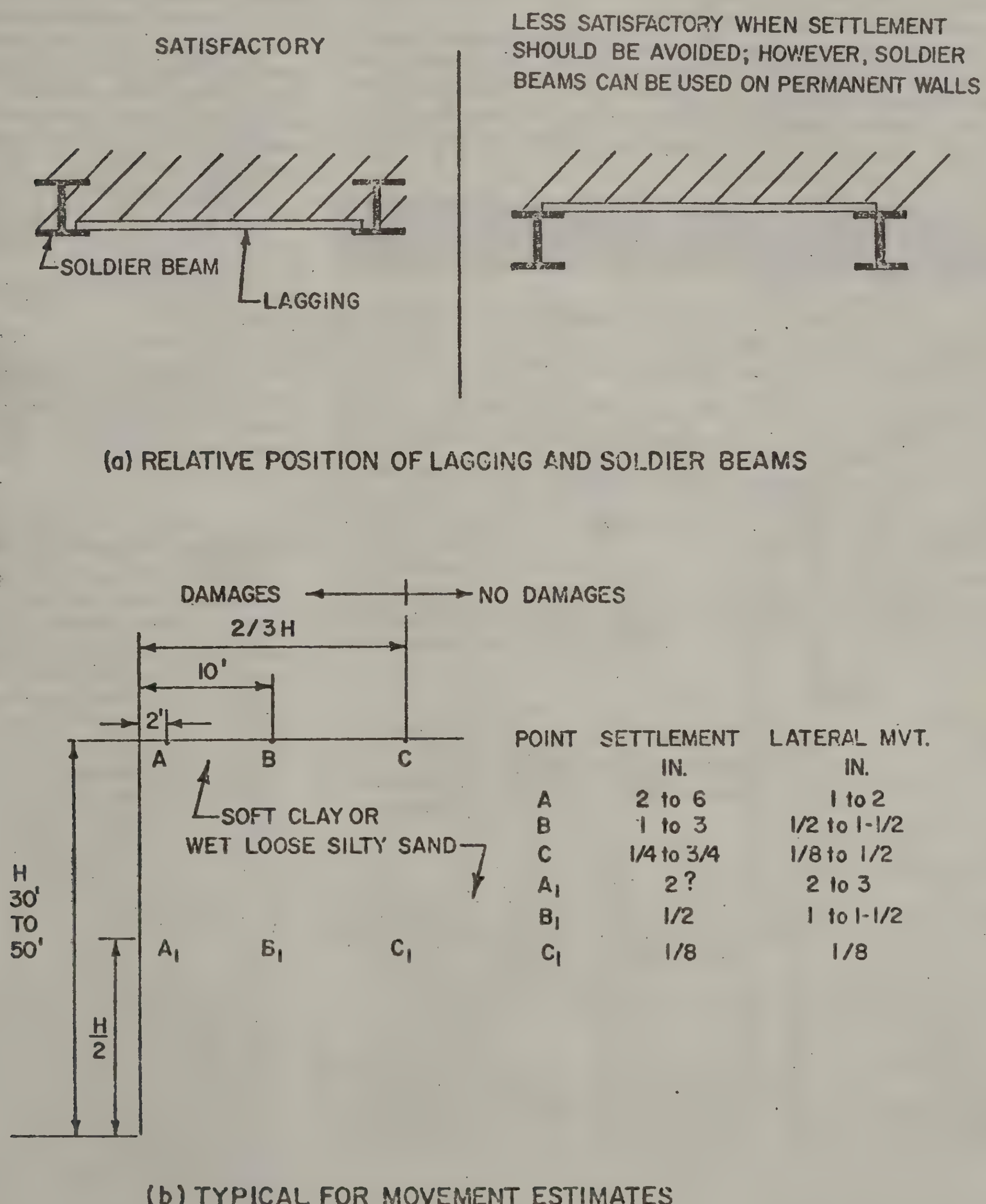


FIGURE 4 RELATIVE POSITION OF LAGGING AND SOLDIER BEAMS AND TYPICAL MOVEMENT ESTIMATES

Chart Estimater Developed For Sheeting

To aid contractors in estimating supports for deep excavations, this chart was prepared by Coakley & Booth, foundation contractors. Theory and analysis of soil pressures for granular soils was worked out by Prof. Donald M. Burmister, head of Columbia University's soil mechanics department in the School of Engineering.

The chart is not applicable for highly plastic clay soil, but can be used for estimating the supporting structure for all soils which can be drained. It is applicable for deep excavations used for buildings, subways, sewers, highways and heavy construction projects. From the chart, the estimator can select the size and spacing of all the structural components: vertical soldier beams, timber sheeting, horizontal wales, and their supporting rakers.

The boundary conditions for soil pressures and the entire analysis for a safe design were worked out by Professor Burmister. The maximum and minimum soil pressures shown on the chart are compiled from a ten-page analysis for all types of granular soils in an unsaturated condition. Structural members are calculated for soil pressures (minimum and maximum) by Contact Sheeting, Inc. engineers, a division of Coakley & Booth.

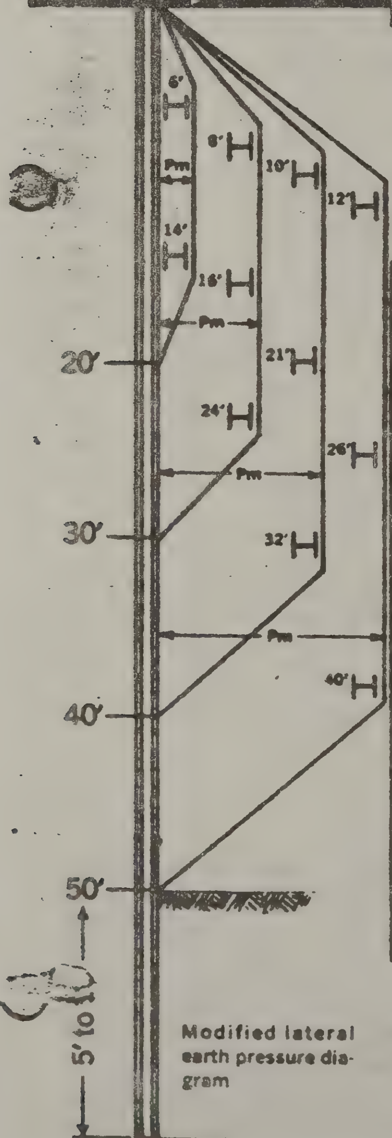
The use of horizontal contact sheeting for supporting deep excavations is advantageous under all conditions where water can be drained from the soil.

Lateral soil pressure intensities (min. & max.) are calculated to cover the extreme range of pressures encountered in free-draining, granular soils for cuts up to 50-ft. in depth. These calculations are based on conservative analyses, in conformance with the latest theories of soil mechanics. A modified trapezoidal earth pressure diagram, in accordance with Dr. Karl Terzaghi's recommendations, is used in determining the soil pressures. Minimum earth pressures and corresponding sizes of members should be used only when previous experiences in an area indicate a coherent, compact, granular soil and good job conditions.

The design given in the table is suggested as a guide for estimating purposes only. The final design should be verified with consulting engineers experienced in foundation design, to assure a safe and economical installation.

The design, using 3" x 10" fir or pine sheeting, supported by fastening assemblies, is predicated on the following:
Allowable stresses (psi)
Steel: f_t=24,000; f_c=10,000; f_v=13,000
Timber: f_c=675
L/r=120; L/d=25
Raker braces spaced 15'-0" (with a maximum rake of 1 on 1½) and soldier beams spaced 7'-6" center to center
Ground water maintained below the bottom sheet

GROUND LEVEL



Modified lateral earth pressure diagram

PREPARED BY

Donald M. Burmister,
Professor of Civil Engineering
Department of Soil Mechanics
School of Engineering, *Columbia University*

DEPTH OF CUT		Pm psf EARTH PRESSURE INTENSITY	SOLDIER BEAMS	WALES (W) & RAKERS (R)
20'	Min.	0	None	None (W) None (R)
	Max.	720	10BP42	12WF65(W) 10BP42(R)
30'	Min.	170	10BP42	10WF49(W) 12"x12"(R)
	Max.	1010	10BP42	14WF84(W) 12BP53(R)
40'	Min.	430	12BP53	12WF65(W) 12BP53(R)
	Max.	1340	14BP73	14WF136(W) 14BP89(R)
50'	Min.	690	12BP74	14WF87(W) 12BP53(R)
	Max.	1660	14WF136	24WF140(W) 14WF150(R)

Sizes listed above are for number and spacing of Wales as shown on pressure diagrams, modified by CS engineers as job experience dictated.

Road Plan Alteration Possible

WASHINGTON, D. C.—Congressional action may be used to get the Bureau of Public Roads to reconsider its insistence on the so-called Wisconsin corridor for a Maryland-District of Columbia Rte. 240 connection. Federal, Maryland and District highway officials stuck to the positions they have held during the controversy.

Senator Francis Case (R-SD) suggested at a Senate Public Roads subcommittee hearing that the agency might be told by Congress to postpone until September an August 1 deadline for submission of revised interstate highway plans to allow the district to justify a route east of Rock Creek Park.

Federal Highway Administrator Bertram D. Tallamy has stated that the major block to any route but the Wisconsin corridor for the Rte. 240 connection would be too expensive and too long for Federal aid. District Highway Director Harold L. Aitken at the Senate subcommittee meeting revealed a map showing that the Wisconsin corridor route would be 9.57 miles long and would cost about $111 million, while the proposed Rock Creek Park proposed route would be about 7.92 miles long and would cost about $109 million.

ENGINEERS' NOTEBOOK

Stability of deep cuts in clay

W. D. LIAM FINN, A.M. ASCE, Assistant Professor
Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

CASE II $\left(B < \frac{D}{3}\right)$

Stability against bottom heave of deep braced cuts in soft clay has received attention in both textbooks and technical journals. Most textbooks that treat this problem present the Terzaghi method[1] for calculating the critical depth, D_c, at which heave will occur.

Bjerrum and Eide[2] drew attention to the fact that the Terzaghi method is not applicable to deep cuts and proposed the formula,

$$D_c = N_c \frac{S}{\gamma}$$

The factor N_c is obtained from charts prepared by Skempton,[3] based on his theory of deep foundations; S is the average shear stress, which may be replaced by c, cohesion; and γ is the unit weight of the soil.

The method presented here is based on predicting the length of surface over which significant shear stresses may be assumed to operate at the instant of incipient heave. The results obtained by this theory will be compared with those given by the formulas of Terzaghi and Bjerrum and Eide, and with the field data reported by Bjerrum and Eide.

The slip surfaces and force system assumed in the Terzaghi method[1] are shown in Fig. 1. Using an ultimate bearing capacity on line AB of 5.7 c, the critical depth is obtained as

$$D_c = \frac{5.7\,c}{\gamma - \frac{c}{b}\sqrt{2}}$$

CASE I
$B > D/3$

At the instant of failure the line AB moves down. Finn[4] has shown that stresses caused by such a displacement are initially dissipated to within a few percent of their maximum value at a distance of $3\,b$ from AB, or about four times the length AB. It is therefore assumed that shear stresses of significant magnitude will not be mobilized over a length of BC greater than $3\,b$.

The net force on AB per unit length of cut, P, is then given by

$$P = \left(\frac{b\sqrt{2}}{2}\,\gamma\,D_c\right) - 3\,bc$$

and the net pressure, p_r, by

$$p_r = \left(\gamma\,D_c\right) - 4.2\,c$$

At the instant of heave, $p_r = 5.7\,c$, the ultimate bearing capacity, and

$$D_c = 9.9\,\frac{c}{\gamma}$$

For convenience, the critical depth is taken as

$$D_c = 10\,\frac{c}{\gamma}$$

Five shafts described by Bjerrum and Eide fall within the range of the above formula ($D \geq 3\,b$). Data on these shafts are given in Table I. Table II gives field results and compares the various theories.

The theory advanced here, like the Terzaghi method, does not take the length of the excavation into account. The length is assumed to be infinite. Thus the results are conservative.

An upper limit is established for the length over which the full shearing resistance may be expected to act at the instant of incipient heave. In practice this length may depend somewhat on the consistency of the clay. There is not sufficient variation in the consistencies of the clays described above to clarify this point.

Correlation with field data appears to be rather good but comparison with data from clays of more widely different characteristics is desirable.

REFERENCES

[1] Terzaghi, K. *Theoretical Soil Mechanics*, John Wiley & Sons, New York, 1943.

[2] Bjerrum, L. and Eide, O. "Stability of Strutted Excavations in Clay," *Geotechnique*, Vol. VI (1956), p. 32.

[3] Skempton, A. W. "The Bearing Capacity of Clays," Building Research Congress, London, 1951, Div. 1, pp. 180-189.

[4] Finn, W. D. "Stresses in Soil Masses under Various Boundary Conditions," Ph.D. Thesis, Univ. of Washington, Seattle, Dec. 1960; also University Microfilms, Univ. of Michigan, Ann Arbor, Mich., Jan. 1961.

TABLE I. Data on deep excavations[2]

Linear dimensions in meters; forces in tons

Shaft Location	b	D	γ	S	N_c	D_c	Type of Failure
1. Goteborg	0.9	25.0	1.54	3.5	9.0	22.8	Total
2. Oslo	1.5	7.0	1.85	1.2	9.0	6.5	Total
3. Trondheim	2.7	19.7	1.80	3.5	8.5	19.5	Total
4. Oslo	1.5	12.0	1.85	2.1	9.0	11.4	Partial
5. Oslo	1.5	11.0	1.93	2.7	9.0	14	No failure

TABLE II. Comparison of theoretical and field results

In shafts of Table I, for depth D_c

Shaft	Terzaghi*	Bjerrum	Finn	Actual (D)
1	..	20.4	22.7	25.0
2	9.5	5.8	6.5	7.0
3	..	17.4	19.4	19.7
4	..	10.2	11.3	12.0
5	..	12.6	14.0	D=11.0†

*The Terzaghi formula yields results only for shaft No. 2.
†This shaft did not fail at a depth of 11 meters.

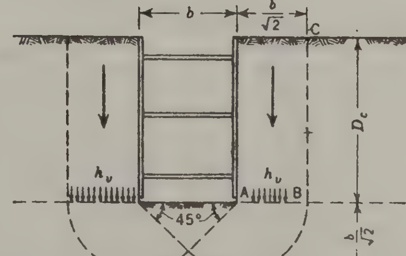


FIG. 1. Assumed mechanism of failure in a deep cut.

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PROJECT: BASE STABILITY OF SHEETED TRENCH

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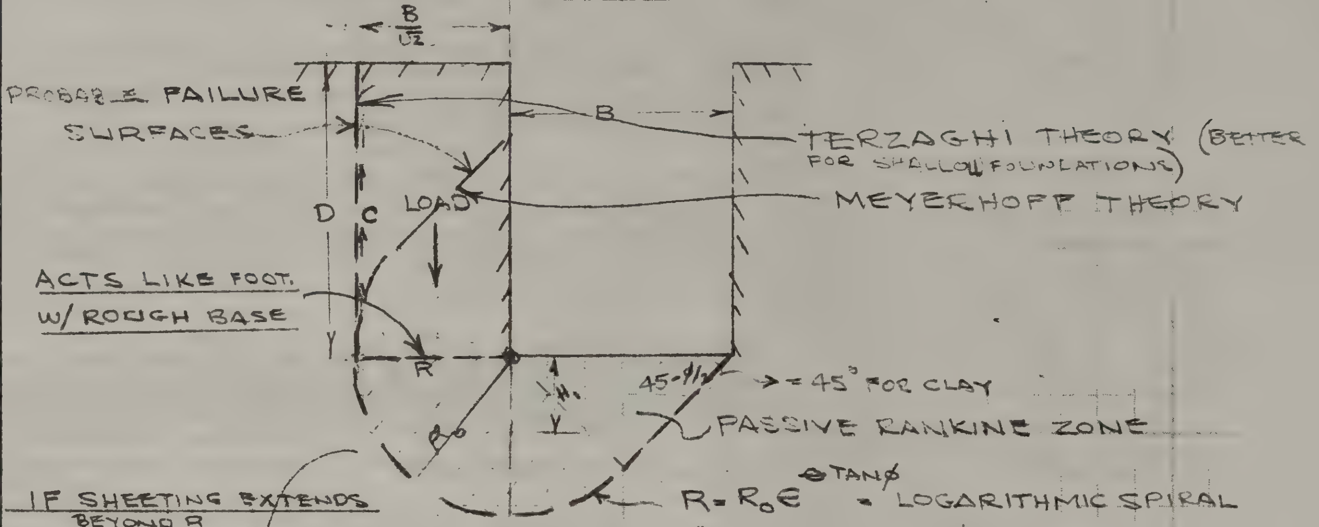
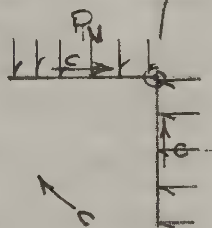
CASE I $(B > D/3)$

TERZAGHI & PECK





$$LOAD = \gamma D \frac{B}{\sqrt{2}} - cD$$

$$P_V = \gamma D - \frac{cD}{B/\sqrt{2}}$$

$$P_{V_{SF=1}} = 5.7c \quad \text{(ROUGH BASE)}$$

$$5.7c = \gamma D_c - \frac{cD_c}{B/\sqrt{2}}$$

$$D_{c_{SF=1}} = \frac{5.7c}{\gamma - \frac{c\sqrt{2}}{B}}$$

$$\frac{5.7c}{\gamma D - \frac{cD}{B/\sqrt{2}}} = \frac{5.7c}{\gamma D - \frac{\sqrt{2}\,cD}{B}} = S.F. \geq 1.5$$

IF S.F. < 1.5 INCREASE SHEETING BEYOND D

$$\Sigma M_o = 0$$

$$P_V \frac{B}{\sqrt{2}} \frac{B}{\sqrt{2}}\left(\frac{1}{2}\right) = P_H \frac{B}{\sqrt{2}} \frac{B}{\sqrt{2}}\left(\frac{1}{2}\right) + c\frac{\pi}{2}\frac{B}{\sqrt{2}}\frac{B}{\sqrt{2}}$$

$$P_V = P_H + \pi c$$

$$P_H = P_V - \pi c$$

H_1 = PENETRATION OF SHEETING

$$P_{\substack{TOTAL\ AGAINST \\ SHEETING}} = (P_V - \pi c)\frac{B}{\sqrt{2}} \xrightarrow{IF} \left(H_1 \geq \frac{2}{3}\frac{B}{\sqrt{2}}\right) \text{ BEC. OF ARCHING}$$

$$= (P_V - \pi c)(1.5 H_1) \xrightarrow{IF} \left(H_1 < \frac{2}{3}\frac{B}{\sqrt{2}}\right)$$

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS — BUREAU OF SOIL MECHANICS

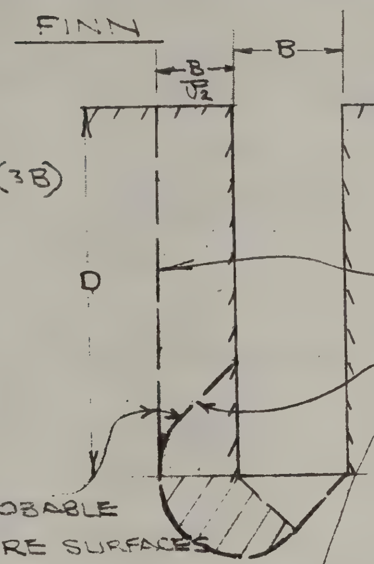
PROJECT: BASE STABILITY OF SHEETED TRENCH — Sheet ...... of ...... Sheets

Prepared by: R.L.G. — Date ......

Checked by: ...... — Date ......

CASE II $\left(B < \frac{D}{3}\right)$

FINN



$D_c = N_c \frac{S}{\gamma}$

$\text{LOAD} = \gamma D \frac{B}{\sqrt{2}} - C(3B)$

$P_v = \gamma D - \frac{C(3B)\sqrt{2}}{B}$

$= \gamma D - 4.24C$

$P_{v_{SF=1}} = 5.7C$

$5.7C = \gamma D - 4.24C$

$D_c = \frac{9.94C}{\gamma}$

$\approx \frac{10C}{\gamma}$

TERZAGHI THEORY

MEYERHOF THEORY
DISREGARDS 4.24C
BETTER FOR DEEP FOUNDATIONS

PROBABLE
FAILURE SURFACES

$\frac{5.7C}{\gamma D - 4.24C} = S.F. \geq 1.5$

NAYCOCK EXCLUDES THIS TERM
FOR DEEP FOUNDATIONS

$D_{SF=1.5} = \frac{\frac{5.7C}{1.5} + 4.24C}{\gamma} = \frac{8.04C}{\gamma} = \frac{8C}{\gamma}$

Form
SM 273 (3/63)

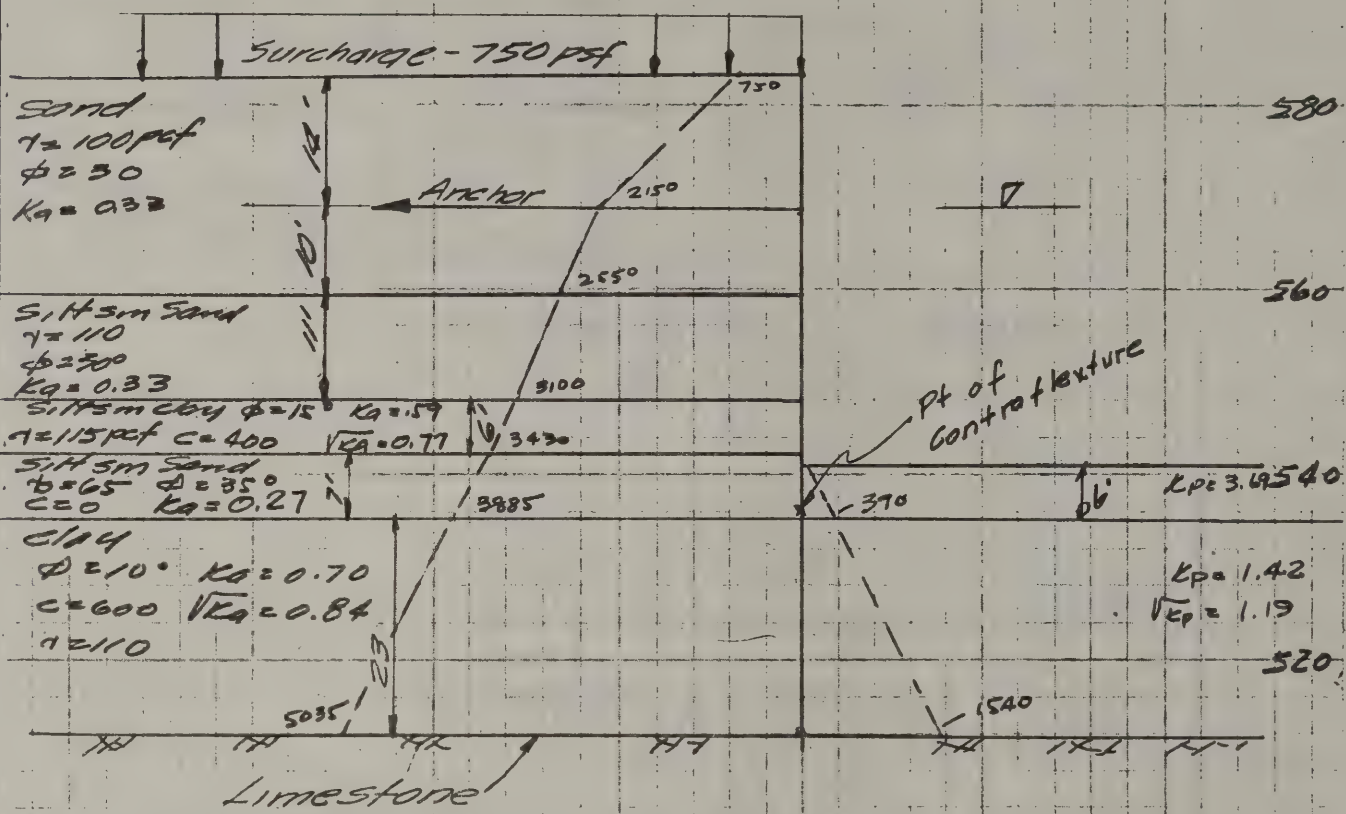
STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS — BUREAU OF SOIL MECHANICS

PROJECT: Niagara Frontier Port Authority - Warehouse B Anchored Bulkhead

Sheet 1 of 9 Sheets
Prepared by: DNG Date 4-6-65
Checked by: RLG Date

Design Values agreed upon @ the 4-6-65 Conference

Vertical Pressures.



Elev 583: pv = 750 psf

Elev 569: pv = 750 + 14(100) = 2150

Elev 559: pv = 2150 + 10(40) = 2550

Elev 548: pv = 2550 + 11(50) = 3100

Elev 542: pv = 3100 + 6(55) = 3430.

Elev 535: pv = 3430 + 7(65) = 3885

Elev 512: pv = 3885 + 23(50) = 5035

Passive
Elev 535: pv = 6(65) = 390 psf
Elev 512: pv = 390 + 23(50) = 1540 psf.

Form M 273 (3/63)

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS BUREAU OF SOIL MECHANICS

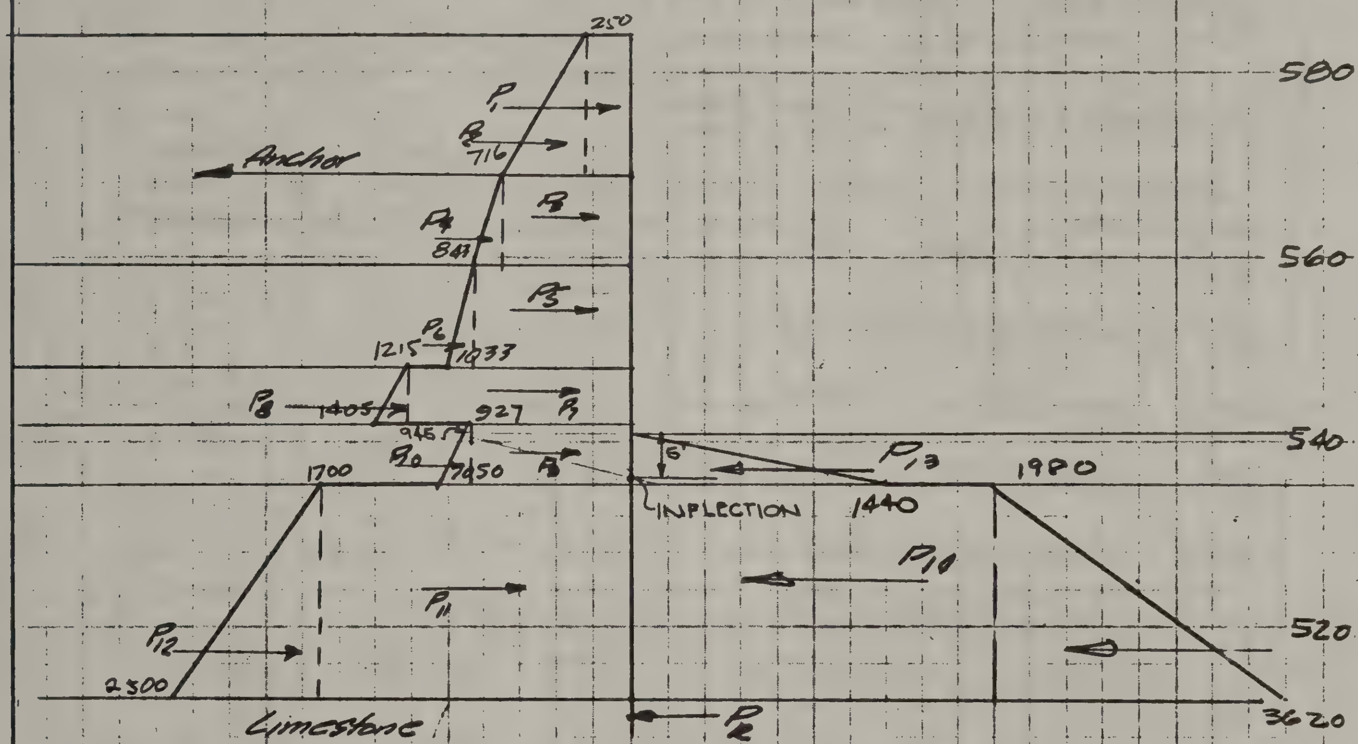
PROJECT: Niagara Frontier Port Authority - Warehouse B Anchored Bulkhead

Sheet 2 of 9 Sheets
Prepared by: DNG Date 4-6-65
Checked by: R.L.G. Date

Horizontal Pressures

Active: $p_a = p_v K_a - 2c\sqrt{K_a}$

Passive: $p_p = p_v K_p + 2c\sqrt{K_p}$



Active:

Elev 583: $p_h = 750(.33) = 250$ psf

Elev 569: $p_h = 2150(.33) = 716$ psf

Elev 559: $p_h = 2550(.33) = 843$ psf

Elev 548: $p_h = 3100(.33) = 1033$ psf
$p_h = 3100(.59) - 800(0.77) = 1215$ psf

Elev. 542: $p_h = 3430(.59) - 800(0.77) = 1405$ psf
$p_h = 3430(0.27) = 927$ psf

Elev 535: $p_h = 3885(.27) = 1050$ psf
$p_h = 3885(.70) - 1200(.84) = 1700$ psf

Elev. 512: $p_h = 5035(.70) - 1200(.84) = 2500$ psf

Form 273 (3/63)

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS BUREAU OF SOIL MECHANICS

PROJECT: Niagara Frontier Port Authority - Warehouse B Anchored Bulkhead
Sheet 3 of 9 Sheets
Prepared by: DNG Date 4-6-65
Checked by: RLG Date

Passive Pressures

Elev. 535: $P_p = 390(3.69) = 1440$ psf
$P_p = 390(1.42) + 1200(1.19) = 1980$ psf

Elev 512: $P_p = 1540(1.42) + 1200(1.19) = 3620$ psf

ΣMoment about tie rod (+)

Force	Arm	Moment
Positive		k-ft
$P_3 = .716(10) = 7.2$	5	36.0
$P_4 = 0.127(10)(1/2) = .63$	6.67	4.2
$P_5 = .843(11) = 9.24$	15.5	143.5
$P_6 = .190(11)(1/2) = 1.05$	17.3	18.2
$P_7 = 1.215(6) = 7.3$	24.0	175.0
$P_8 = .190(6)(1/2) = .57$	25.0	14.2
$P_9 = .927(7) = 6.5$	30.5	198.0
$P_{10} = .123(7)(1/2) = .43$	31.7	13.7
$P_{11} = 1.7(23) = 39.2$	45.5	1780.0
$P_{12} = .8(23)(1/2) = 9.2$	49.3	454.0
		$\Sigma = 2836.8^{k\text{-}ft}$ ✓
Negative		
$P_1 = .250(14) = 3.5$	7	24.5
$P_2 = .466(14)1/2 = 3.3$	4.67	15.5
$P_{13} = 1.44(6)(1/2) = 4.3$	32.0	138
$P_{14} = 1.98(23) = 45.5$	45.5	2070
$P_{15} = 1.64(23)1/2 = 18.8$	49.3	930
		$\Sigma = 3178.0$ ✓

Form 273 (3/63)

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS BUREAU OF SOIL MECHANICS

PROJECT: Niagara Frontier Port Authority – Warehouse "B" Sheeting Analysis

Prepared by: DNG Date 4-8-65

Checked by: Date

If $P_R = 0$ (No resistance to sheeting from rock)

then $S.F = \frac{3178}{2837} = \underline{\underline{1.12}}$ If $P_R = 0$

Friction developed at the bottom of the sheet between the sheeting and rock.

Use a friction of 0.25 ($\delta = 14°$) for sand, and sand & silt against steel.

$\Sigma(P_1 + P_2 + P_3 + P_4 + P_5 + P_6 + P_7 + P_8 + P_9 + P_{10})0.25$

$\Sigma(P'_5)(0.25) = 39.6(.25) = 9.9^k$

$P_{13}(0.25) = 4.3(.25) = 1.1^k$

Net $= 9.9 - 1.1 = 8.8^k$ ← This is the net vertical force on the sheeting from the friction between the soil and sheeting.

Wt of sheeting / Lin ft (For ZP38)

$W = 38^{\#} \times 71 = 2{,}700^{\#} = 2.7^k$

Total Vertical load $= 8.8 + 2.7 = 11.5^k$

Using a Coef. of friction of 0.5 between tip of sheeting and rock.

$P_R = 11.5(0.5) = 5.75^k$

Additional resisting moment

$5.75 \times 57' = 328^{k\text{-}ft}$

$SF = \frac{3178 + 328}{2837} = \frac{3506}{2837} = \underline{\underline{1.24}}$ If $P_R = 5.75$

Form 273 (3/63)

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS BUREAU OF SOIL MECHANICS

PROJECT: NIAGARA FRONTIER PORT AUTHORITY - WAREHOUSE B ANCHORED BULKHEAD

Sheet 5 of 9 Sheets

Prepared by: R.L.G. Date 4-9-65

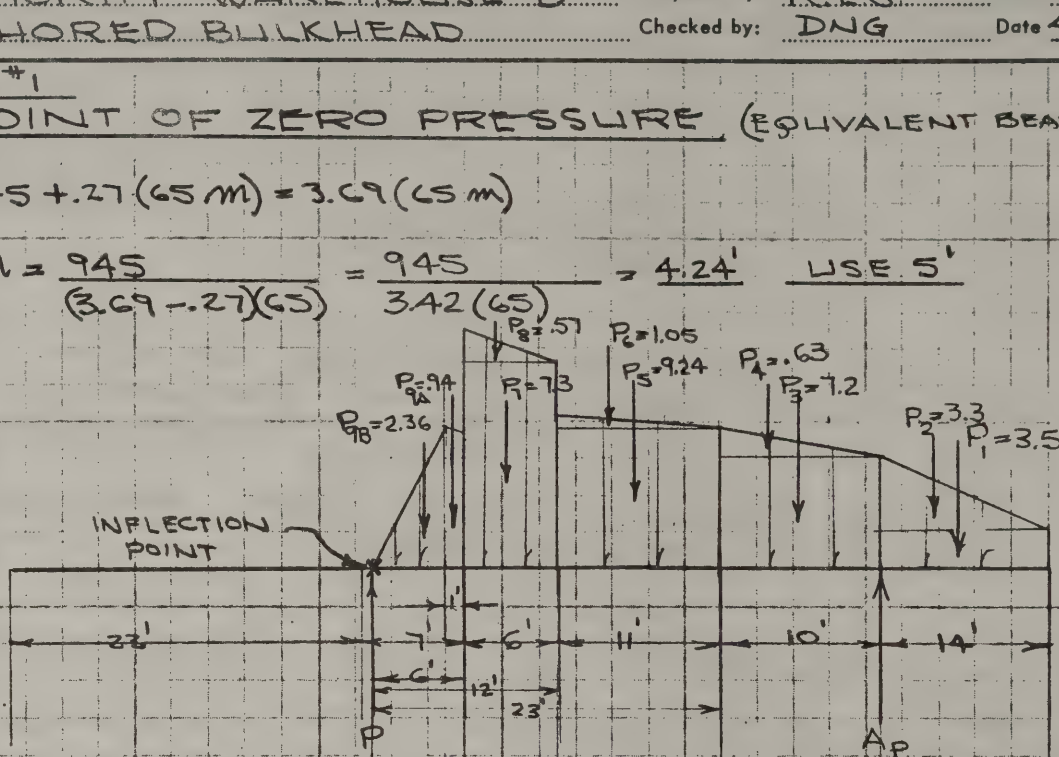
Checked by: DNG Date 4-9-65

CASE #1

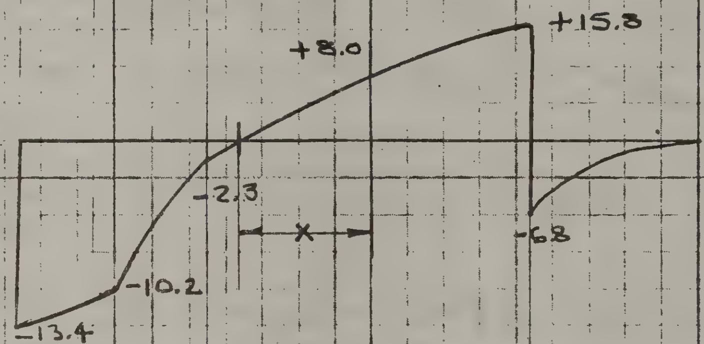
POINT OF ZERO PRESSURE (EQUIVALENT BEAM)

$945 + .27(65\,m) = 3.69(65\,m)$

$m = \frac{945}{(3.69-.27)(65)} = \frac{945}{3.42(65)} = \underline{4.24'}$ USE 5'



SHEAR



$-.84X - .0\frac{174}{2}X^2 = +8.0$ $(X+48.3)^2 = 920 + 2330 = 3250 = 57^2$

$X^2 + 96.6X = +920$ $X = 57 - 48.3 = \underline{8.7'}$

$\underline{A_P} = \frac{40}{33}(3.5) + \frac{37.7}{33}(3.3) + \frac{28}{33}(7.2) + \frac{26.3}{33}(.63) + \frac{17.5}{33}(9.24) + \frac{15.7}{33}(1.05) + \frac{9}{33}(7.3)$

$+ \frac{8}{33}(.57) + \frac{5.5}{33}(.94) + \frac{3.3}{33}(2.36) = 4.24 + 3.77 + 6.11 + .50 + 4.89 + .50 + 1.99$

$+ .14 + .16 + .24 = \underline{22.6}$ K/FT

$\underline{P} = \frac{29.7}{33}(2.36) + \frac{27.5}{33}(.94) + \frac{25}{33}(.57) + \frac{24}{33}(7.3) + \frac{15.5}{33}(9.24) + \frac{17.3}{33}(1.05) + \frac{5}{33}(7.2)$

$+ \frac{6.7}{33}(.63) - \frac{7}{33}(3.5) - \frac{4.7}{33}(3.3) = 2.12 + .78 + .43 + 5.15 + 4.33 + .55 + 1.09$

$+ .13 - .74 - .47 = \underline{13.41}$ K/FT

$\underline{22.6 + 13.4 = 36.0}$ CHECK

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS — BUREAU OF SOIL MECHANICS

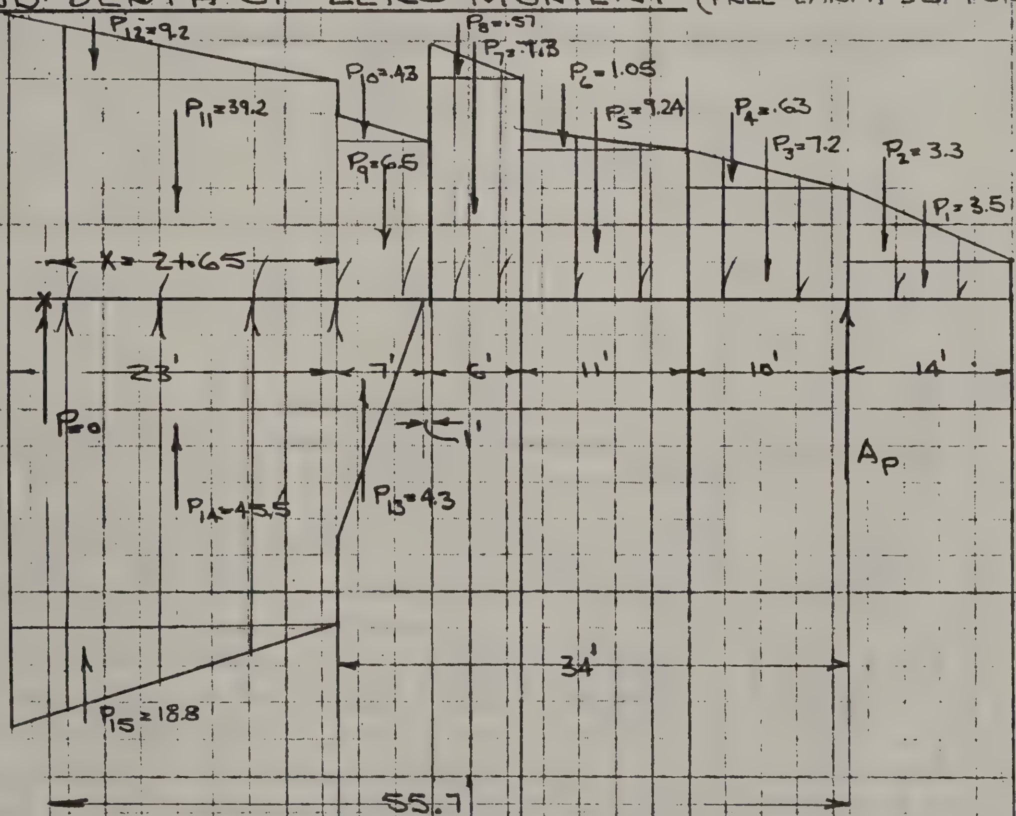
PROJECT: NIAGARA FRONTIER PORT AUTHORITY - WAREHOUSE B ANCHORED BULKHEAD
Sheet 6 of 9 Sheets
Prepared by: RLG Date 4-9-65
Checked by: DNG Date

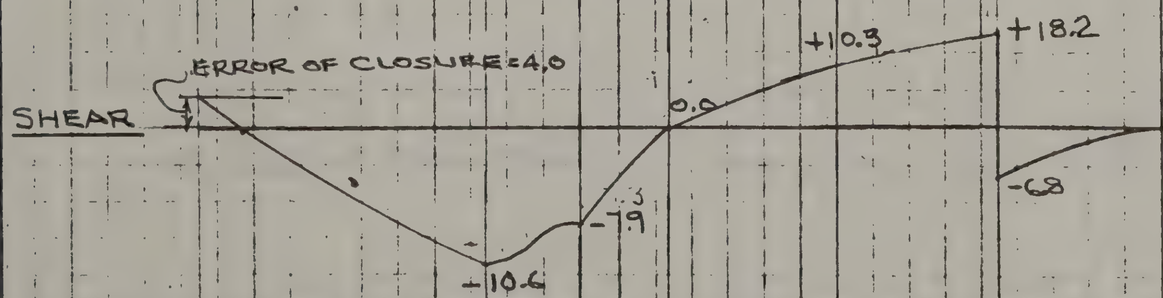
MAX. BENDING MOMENT

$$M_{MAX.} = 3.5(25.7) + 3.3(23.4) + 7.2(13.7) + .63(12) + .84(8.7)\left(\frac{8.7}{2}\right) + .0174\left(\frac{8.7}{2}\right)^2 \frac{8.7}{3}$$
$$- 22.6(18.7) = 90 + 77.2 + 98.6 + 7.6 + 31.8 + 1.9 - 422$$
$$\approx \underline{115\ 'K}$$

CASE #2

FIND DEPTH OF ZERO MOMENT (FREE EARTH SUPPORT)





Form
M 273 (3/63)

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS BUREAU OF SOIL MECHANICS

PROJECT: NIAGARA FRONTIER PORT AUTHORITY - WAREHOUSE B ANCHORED BULKHEAD
Sheet 7 of 9 Sheets
Prepared by: RLG Date 4-9-65
Checked by: DNG Date

$\Sigma M_{A_p} = 0$

FROM PAGE 3

ΣM_{A_p} OF $P_3 \rightarrow P_{10} = 602.8$

ΣM_{A_p} OF $P_{13}, P_1, P_2 = \underline{178.0}$

ΣM_{A_p} OF $P_1 \rightarrow P_{10}, P_{13} = \underline{424.8'K}$

$$\Sigma M_{A_p} = 424.8 + 1.70\, x\left[34+\frac{x}{2}\right] + .0348\frac{x^2}{2}\left[34+\frac{2}{3}x\right]$$
$$-1.98\, x\left[34+\frac{x}{2}\right] - .0712\frac{x^2}{2}\left[34+\frac{2}{3}x\right] = 0$$
$$424.8 - .28x\left[34+\frac{x}{2}\right] - .0364\frac{x^2}{2}\left[34+\frac{2}{3}x\right] = 0$$
$$424.8 - 9.52x - .14x^2 - .062x^2 - .0121x^3 = 0$$
$$x^3 + 16.7x^2 + 787x - 35100 = 0$$
$$\underline{x} = \underline{21.65}$$

$\Sigma M_p = 0$

$$\underline{A_p} = -.0364\frac{(21.65)^2}{2}\left(\frac{7.18}{55.65}\right) - .28(21.65)\left(\frac{10.80}{55.65}\right) - 4.3\left(\frac{23.65}{55.65}\right) + .43\left(\frac{23.95}{55.65}\right) + 6.5\left(\frac{24.7}{55.65}\right)$$
$$+ .57\left(\frac{30.2}{55.65}\right) + 7.3\left(\frac{31.2}{55.65}\right) + 1.05\left(\frac{37.9}{55.65}\right) + 9.24\left(\frac{39.7}{55.65}\right) + .63\left(\frac{48.5}{55.65}\right) + 7.2\left(\frac{50.2}{55.65}\right)$$
$$+ 3.3\left(\frac{59.9}{55.65}\right) + 3.5\left(\frac{62.2}{55.65}\right) = -1.10 - 1.16 - 1.83 + .18 + 2.88 + .31 + 4.08$$
$$+ .71 + 6.57 + .55 + 6.48 + 3.54 + 3.90$$
$$= \underline{\underline{25.1}}\ K/FT$$

$$\underline{M_{MAX}} = -25.1(21) + 1.05(3.7) + 9.24(5.5) + .63(11.3) + 7.2(16) + 3.3(25.7) + 3.5(28)$$
$$= -527 + 3.9 + 50.9 + 7.1 + 115 + 85 + 98 = \underline{\underline{167'K}}$$

Form 73 (3/63)

STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS BUREAU OF SOIL MECHANICS

PROJECT: NIAGARA FRONTIER PORT AUTHORITY-WAREHOUSE B ANCHORED BULKHEAD

Prepared by: RLG Date 4-9-65

Checked by: Date

USE ZP 38 $S_x = 46.8 \text{ in}^3$

EQUIVALENT BEAM

$M = 115\,'K$ $S_Y = 45\ K/in^2$ $S_{ALL} = .6(45) = 27\ K/in^2$

$P_A = 22.6\ K/FT$

$S_B = \frac{M}{S} = \frac{115(12)}{46.8} = 29.5\ K/in^2 > 27$ N.G

$3''\phi$ TIE RODS ($A = 7.07\ in^2$) $6'$ c/c $S_Y = 33\ K/in^2$, $S_{ALL} = 20\ K/in^2$

$P_A = 22.6(6) = 135.5\ K$

$S_A = \frac{135.5}{7.07} = 19.15\ K/in^2 < 20\ K/in^2$ O.K.

FREE EARTH SUPPORT

$M = 167\,'K$

$P_A = 25.1\ K/FT$

$S_B = \frac{167(12)}{46.8} = 42.8$ KSI < 27 KSI NG

$P_A = (25.1)(6) = 151\ K$

$S_A = \frac{151}{7.07} = 21.3$ KSI > 20 KSI NG

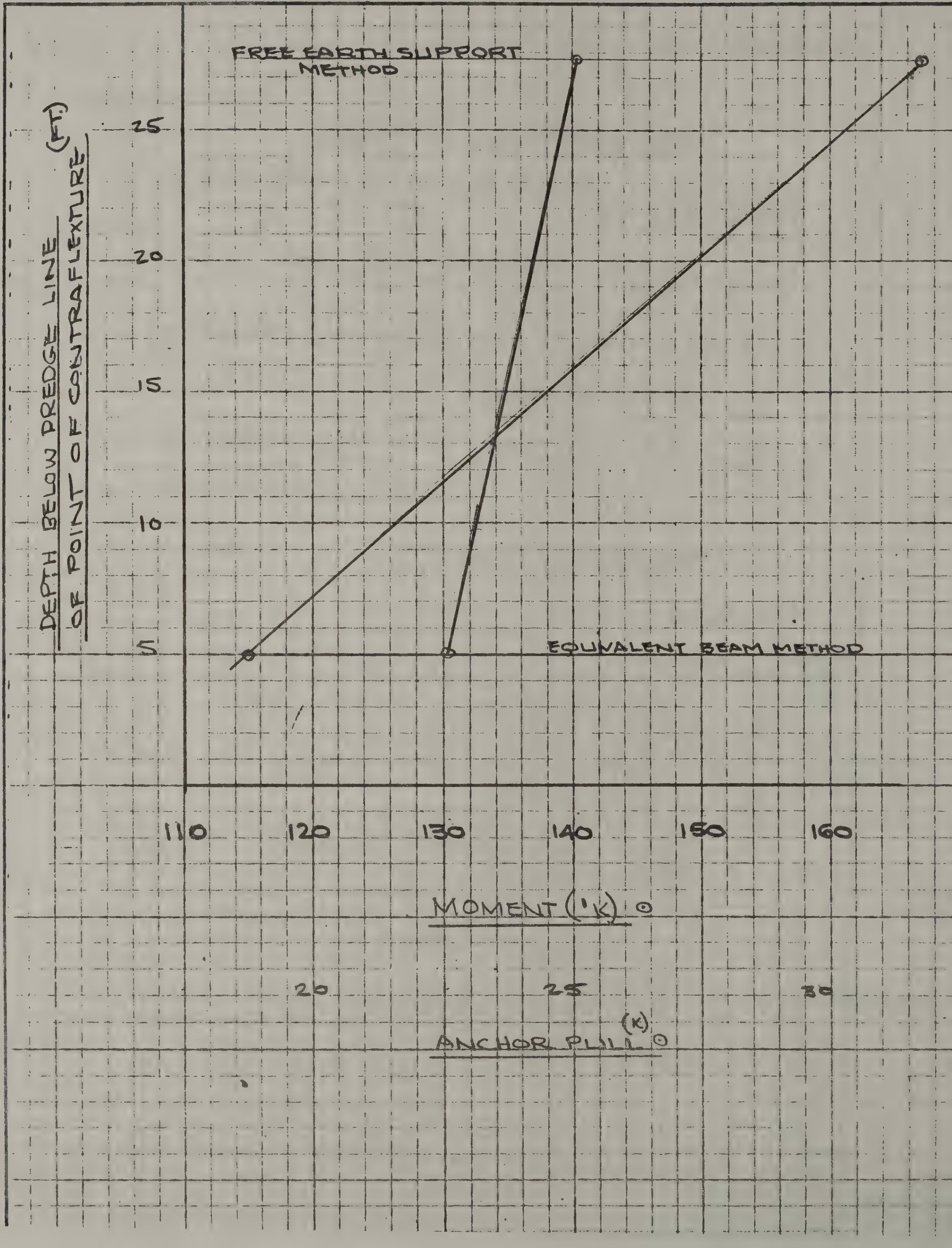
STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS · BUREAU OF SOIL MECHANICS

PROJECT: NIAGARA FRONTIER PORT AUTHORITY - WAREHOUSE B ANCHORED BULKHEAD

Sheet 9 of 9 Sheets

Prepared by: RLG Date 4-9-65

Checked by: Date



1. Introduction

These notes outline some of the important design considerations that effect the earth pressures behind retaining walls. The services of the Bureau of Soil Mechanics are available for any problems concerning earth pressures and bearing capacity in retaining wall design.

The lateral earth pressure is the pressure on the back of a retaining wall caused by a soil mass behind the wall. The force exerted on the rear of the wall is the force required to prevent the mass of soil from sliding along a sloping surface called the failure plane. This is shown in Figure 1.

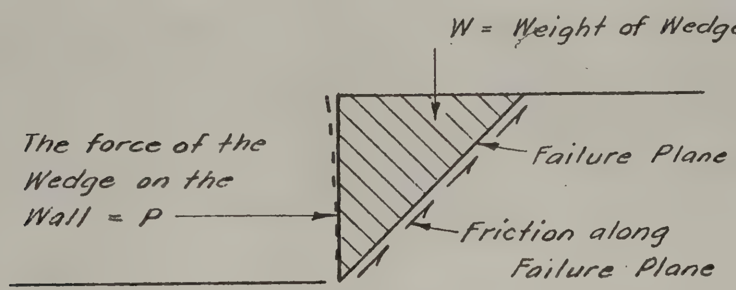


FIGURE 1

2. Conditions to Develop Wall Pressures

The wedge of soil in Figure 1, is held in place by the force P and the friction along the failure plane. In order to develop the friction along the plane there must be some small movement.

If the wall deflects a small amount then the friction will be mobilized and the force P will be smaller than if no movement is allowed. This case is called the active pressure condition where the friction helps in reducing the pressures on the wall.

If the wall is pushed back then the force on the wall must be large enough to resist the friction along the plane. This case is called the passive pressure condition where the frictional resistance increases the pressure on the wall.

Extensive model tests by Terzaghi and Tschebotarioff have provided the information shown in Figure 2.

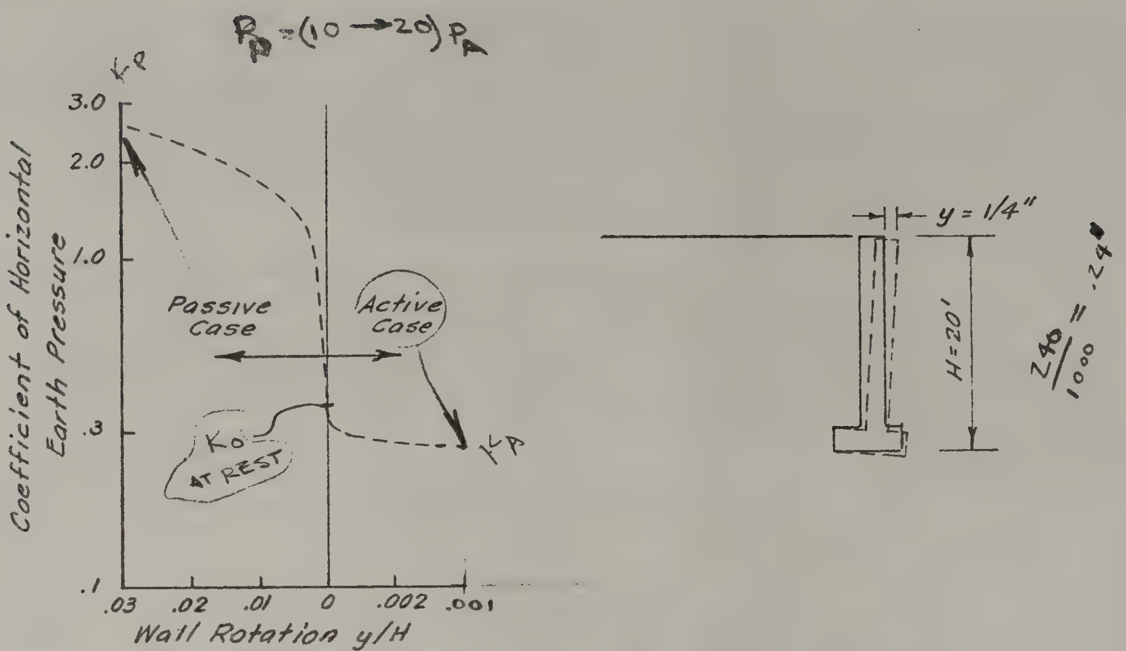


FIGURE 2

For a 20 ft. wall, a movement at the top of 1/4 inch $\approx \frac{1}{1000}$ HEIGHT OF WALL is required to reduce the pressure to the active condition. If the wall did not deflect, the pressures on the wall would be 2 to 3 times greater and if the wall is pushed back the pressures on the wall would be 8 to 10 times larger than the pressures for the active condition.

Standard design curves in use today assume that the wall will be permitted to deflect a small amount to permit the active pressure case. This has proved to be practical and satisfactory since the deflection is so small it is usually not even observed.

3. Use of the Equivalent Fluid Pressure Method

Soil, unlike water, has a shear strength and the ratio of the lateral pressure to the vertical pressure is not equal to one as is the case for water. For soils, the lateral pressure is taken as the vertical pressure multiplied by a lateral pressure coefficient, K_a. The lateral pressure coefficient is, among other things, a function of the strength of the soil. The friction angle, ϕ is a measure of the shear strength of a granular soil. In equation form this is as follows:

$ph = K_a\ pv$

Where: ph = the lateral pressure

pv = the vertical pressure

K_a = the lateral pressure coefficient

$K_a = \frac{1-\sin\phi}{1+\sin\phi}$ (for a horizontal backfill)

Where: ϕ = the friction angle of soil

In the equivalent fluid pressure method, the soil is assumed to behave as a liquid. However, the unit weight of the soil must be modified to account for its strength. This is easily done by multiplying the weight of the soil by the coefficient of lateral pressure, K_a, and arriving at a new weight which is called the equivalent fluid weight or the equivalent fluid pressure.

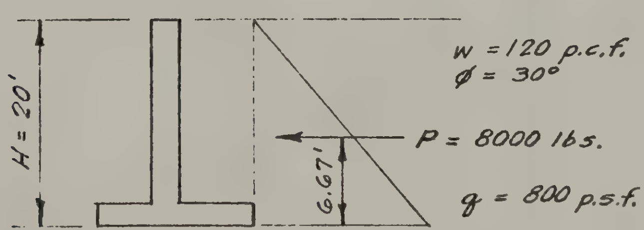
$we = K_a\ w$

Where: we = the equivalent fluid pressure

w = the unit weight of the soil

4. Method of Design

An example of this method of design is shown in Figure 3.



$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{0.5}{1.5} = 0.333$$

$$\gamma_e = K_a(\gamma) = 0.333\,(120) = 40 \text{ p.c.f.}$$

The pressure at the bottom of the wall $q = 20(40) = 800$ p.s.f.

$$P = 800\left(\frac{20}{2}\right) = 8000 \text{ lbs.}$$

or

$$P = \frac{\gamma_e H^2}{2} = \frac{40\,(20)^2}{2} = 8000 \text{ lbs.}$$

FIGURE 3

To facilitate the determination of equivalent fluid pressures for the sloping and broken slope backfill condition, charts entitled, "Pressures on Walls Supporting Embankments" Drawings SM 1661 A, B & C have been prepared and attached to these notes. The assumptions used in preparing the charts and their limitations are listed on Drawing SM 1661A.

The above method, RANKINE THEORY and charts are not valid when the backfill is not a homogenous granular material and/or a compacted embankment material. In addition, this method or these charts should not be used to determine pressures on crib walls, bin walls and gravity walls which have sloping backs and no projecting heels. (TER. & PECK P145)
IN THIS CASE USE COULOMB'S THEORY (NAVDOCK'S PAGES 7-10-5 & 7-10-6)

5. Factors Which Effect the Pressures on the Wall

A. The friction angle of the soil - As shown in Figure 4, the equivalent fluid pressure, we, decreases as the friction angle increases.

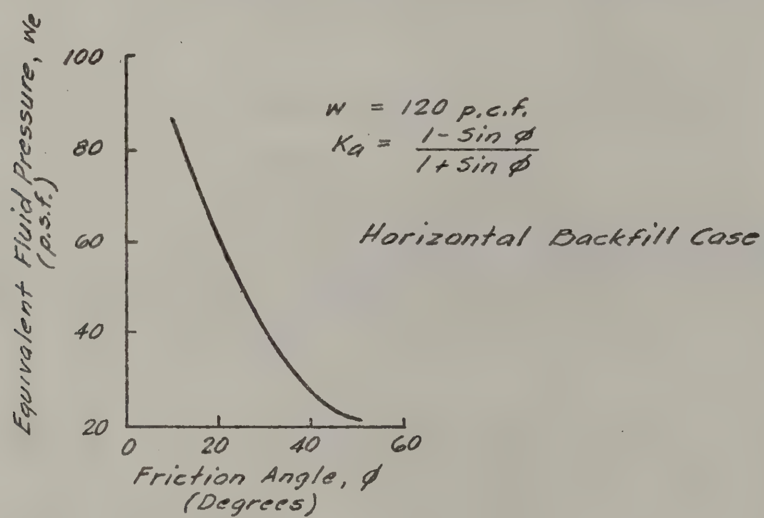


FIGURE 4

B. Effect of soil density on pressures - The more dense the wall backfill material, the higher the friction angle. This is shown in Figure 5.

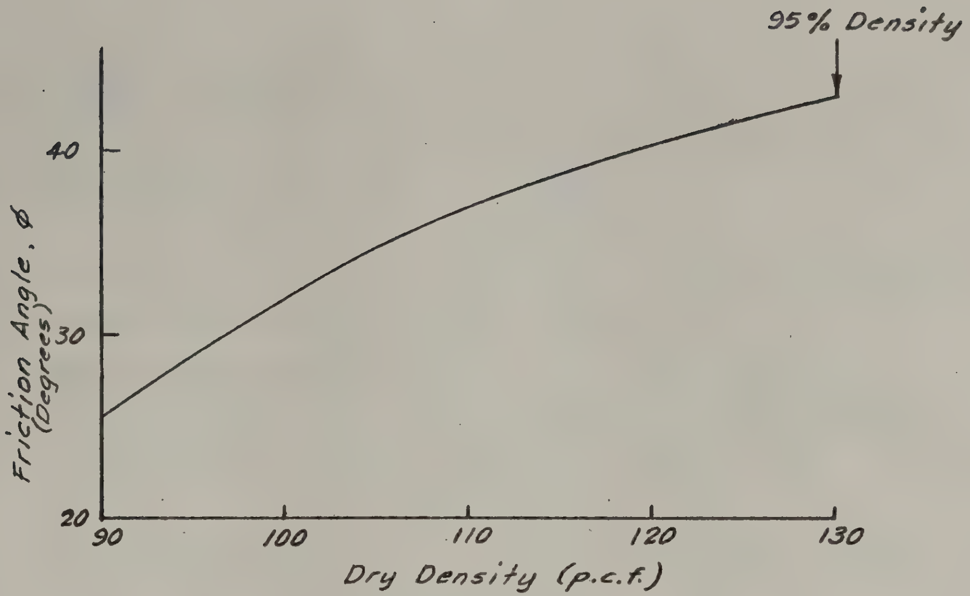


FIGURE 5

C. Slope of backfill behind wall - As the slope of the backfill behind the wall increases the equivalent fluid pressure increases. This is shown in Figure 6.

RANKINE'S EQUATION

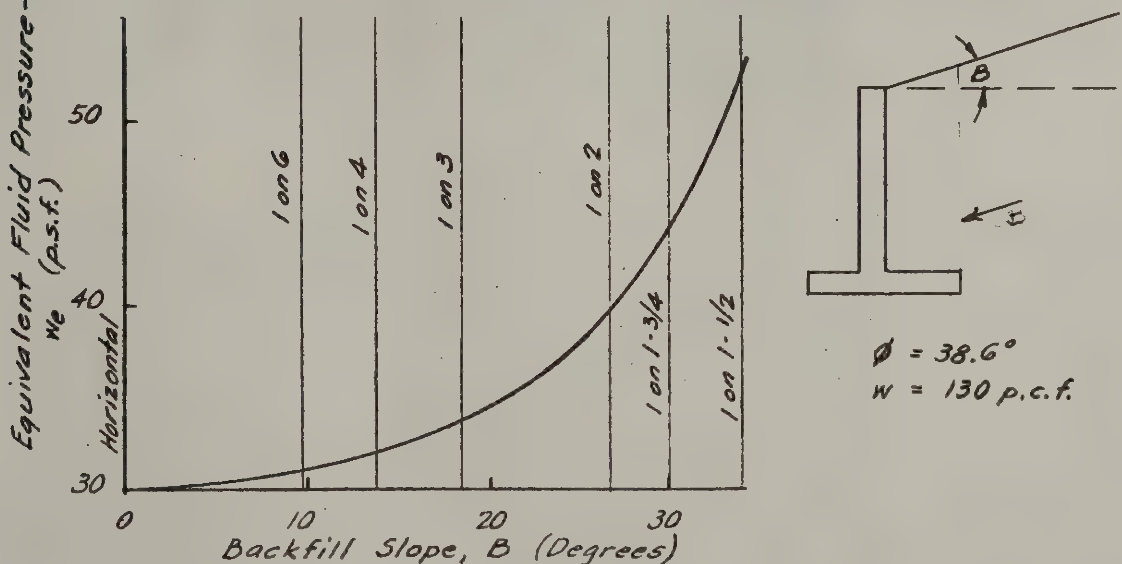
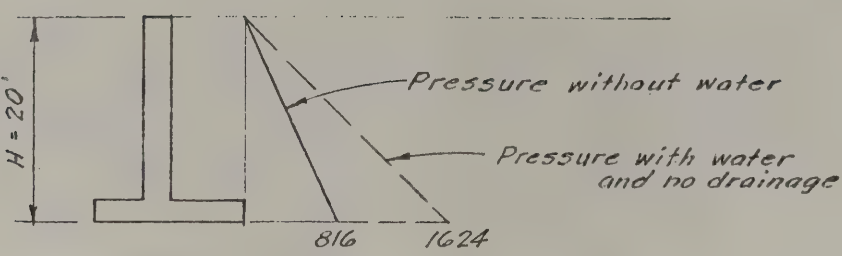


FIGURE 6

D. Drainage - If there is not adequate drainage behind the wall, then the water collecting behind the wall will create an added pressure on the rear of the wall. Figure 7 is an example of the high pressures caused by water.



Unit weight of soil, w, = 122.4 p.c.f.
Submerged (Buoyant) unit weight of soil = 122.4 - 62.4 = 60 p.c.f.
Friction angle from testing, $\phi = 30°$, $K_a = 0.333$

Pressures without water

$w_e = 122.4 \times .333 = 40.8$ p.c.f.

$q = 40.8\,(20) = 816$ p.s.f.

Pressures with water

$w_e = 60 \times .333 + 62.4 = 82.4$ p.c.f.

$q = 82.4\,(20) = 1648$ p.s.f.

FIGURE 7

E. Effect of slope of back of wall - For crib walls, bin walls or gravity walls which do not have a projecting heel, the slope of the back of the wall has a large effect on the coefficient of lateral pressure, K_a, which in turn determines the soil pressure on the rear of the wall. Figure 8 shows this effect.

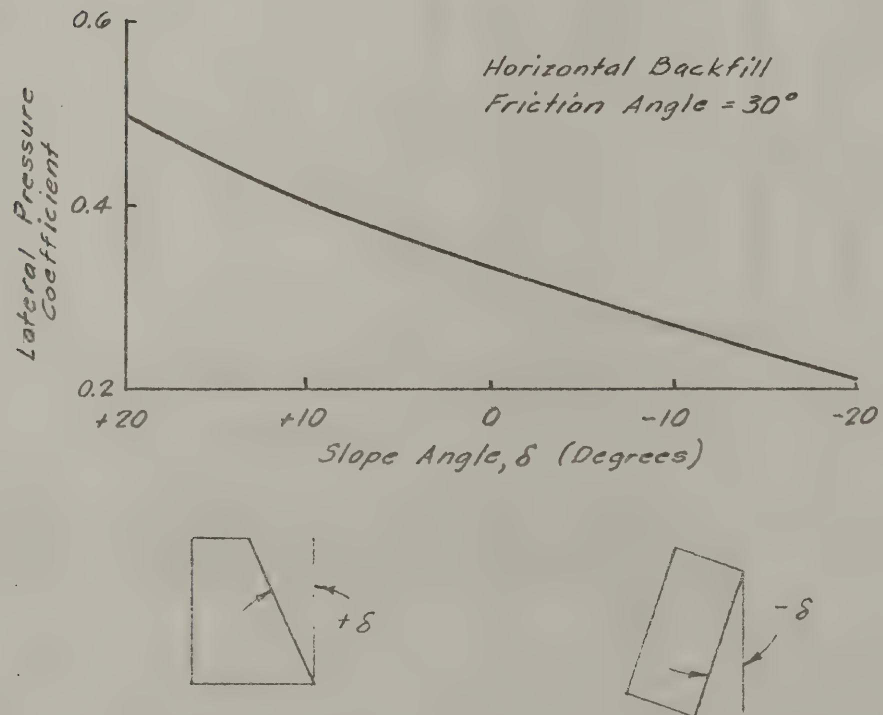


FIGURE 8

F. Backfill material gradation and particle shape - A higher density and a greater friction angle can be obtained from a well graded material composed of angular particles as compared to a material composed of rounded particles of uniform size.

6. <u>Check Lists of Soil Considerations in Retaining Wall Design</u>

   A. Obtain adequate soil explorations.

   B. Does the wall support natural soils, i.e. cuts? If so, investigate the strength of natural soils because earth pressures may be higher than indicated on charts for walls supporting embankments.

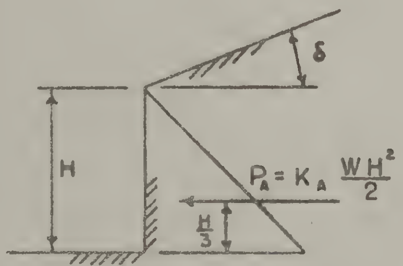
   C. Are the foundation soils beneath the wall adequate for spread footings or will piles be required?

   D. Is the wall drainage adequate? Walls supporting wet cuts where large volumes of water are encountered may require special drainage measures.

   E. Will the wall be backfilled with a properly compacted granular material?

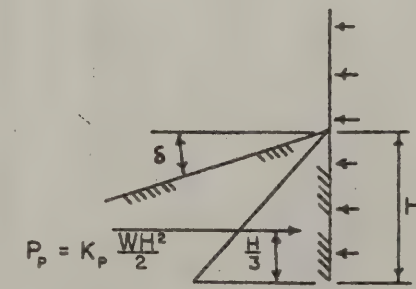
   F. Will there be any temporary or permanent surcharges behind the wall?

ACTIVE EARTH PRESSURE



W = UNIT WEIGHT OF SOIL
Φ = ANGLE OF INTERNAL FRICTION

PASSIVE EARTH PRESSURE



W = UNIT WEIGHT OF SOIL
Φ = ANGLE OF INTERNAL FRICTION

SAME FOR THESE CONDITIONS

$$K_A = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\delta)}}\right]^2$$

(ALSO IN FIG. 5-11, 5-12 NAVDOCKS)

$$K_P = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\delta)}}\right]^2$$

COULOMB'S EQUA. 0 WALL FRICTION WALL = VERTICAL

$$K'_A = K_A\cos\delta = \cos^2\delta\,\frac{\cos\delta-\sqrt{\cos^2\delta-\cos^2\phi}}{\cos\delta+\sqrt{\cos^2\delta-\cos^2\phi}}$$

$$K'_p = K_p\cos\delta = \cos^2\delta\,\frac{\cos\delta+\sqrt{\cos^2\delta-\cos^2\phi}}{\cos\delta-\sqrt{\cos^2\delta-\cos^2\phi}}$$

RANKINE EQUA

TABLE OF VALUES OF K_A

δ		Φ IN DEGREES							
DEG	SLOPE	10	15	20	25	30	35	40	45
0	0	.704	.589	.490	.406	.333	.271	.217	.172
5	1:11.5	.769	.635	.524	.431	.352	.284	.227	.178
10	1:5.7	.970	.704	.569	.453	.374	.300	.238	.186
15	1:3.7		.933	.639	.505	.402	.319	.251	.194
20	1:2.7			.883	.572	.441	.344	.270	.204
25	1:2.1				.821	.505	.378	.288	.217
30	1:1.7					.750	.436	.318	.235
35	1:1.4						.671	.369	.260
40	1:1.2							.587	.304
45	1:1.0								.500

TABLE OF VALUES OF K_P

δ		Φ IN DEGREES							
DEG.	SLOPE	10	15	20	25	30	35	40	45
0	0	1.420	1.698	2.039	2.464	3.000	3.690	4.599	5.829
5	1:11.5	1.262	1.504	1.792	2.144	2.577	3.124	3.826	4.747
10	1:5.7	.970	1.295	1.551	1.925	2.224	2.644	3.193	3.897
15	1:3.7		.933	1.299	1.566	1.866	2.223	2.660	3.204
20	1:2.7			.883	1.278	1.553	1.848	2.118	2.629
25	1:2.1				.821	1.230	1.501	1.796	2.140
30	1:1.7					.750	1.162	1.428	1.712
35	1:1.4						.671	1.076	1.331
40	1:1.2							.587	.972
45	1:1.0								.500

REFERENCE: SUBSTRUCTURE DESIGN AND ANALYSIS
BY PAUL ANDERSEN

LATERAL EARTH PRESSURE COEFFICIENTS
(Zero wall friction)

PRESSURES ON RETAINING WALLS SUPPORTING EMBANKMENTS

Use of Earth Pressure Charts

Figure 2 gives the equivalent horizontal fluid earth pressure, P_h, for various slopes and h/H ratio. Figure 3 gives the vertical equivalent fluid earth pressure, P_v, for the same conditions.

Assumptions

The following soil properties were used in preparing these curves. The backfill is compacted material with a friction angle of 38.6 degrees and a unit weight of 130 lbs. per cubic ft.

Discussion

These curves are only applicable for retaining walls supporting fills composed of compacted embankment materials. For retaining walls supporting natural soil, an investigation should be made to determine if the strength of the natural soil is less than the above assumed strength. If this is the case then these curves should not be used as the resulting soil pressures may be greater.

SHEET 1 OF 3

PREPARED BY: D. N. Geoffroy	APPROVED DATE: 12/17/64 Wm. P. Hofmann PRINCIPAL SOILS ENGINEER
SUBMITTED FOR APPROVAL BY: Lyndon H. Moore 12/16/64	

PRESSURES ON WALLS SUPPORTING EMBANKMENTS	FIGURE 1	STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS DIVISION OF CONSTRUCTION BUREAU OF SOIL MECHANICS

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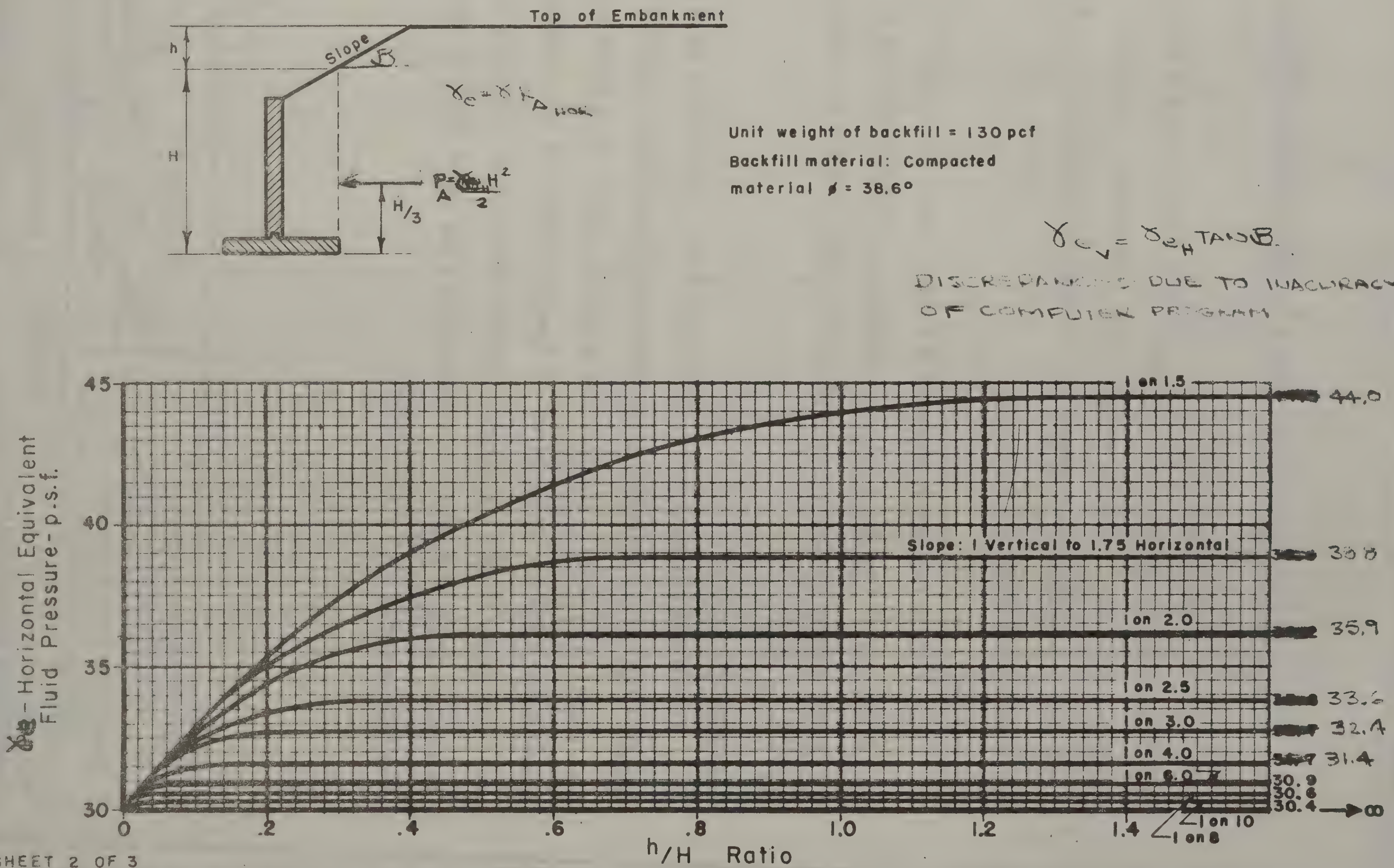

SHEET 2 OF 3

PRESSURES ON WALLS SUPPORTING EMBANKMENTS	EQUIVALENT HORIZONTAL FLUID PRESSURE	FIGURE 2	STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS DIVISION OF CONSTRUCTION BUREAU OF SOIL MECHANICS

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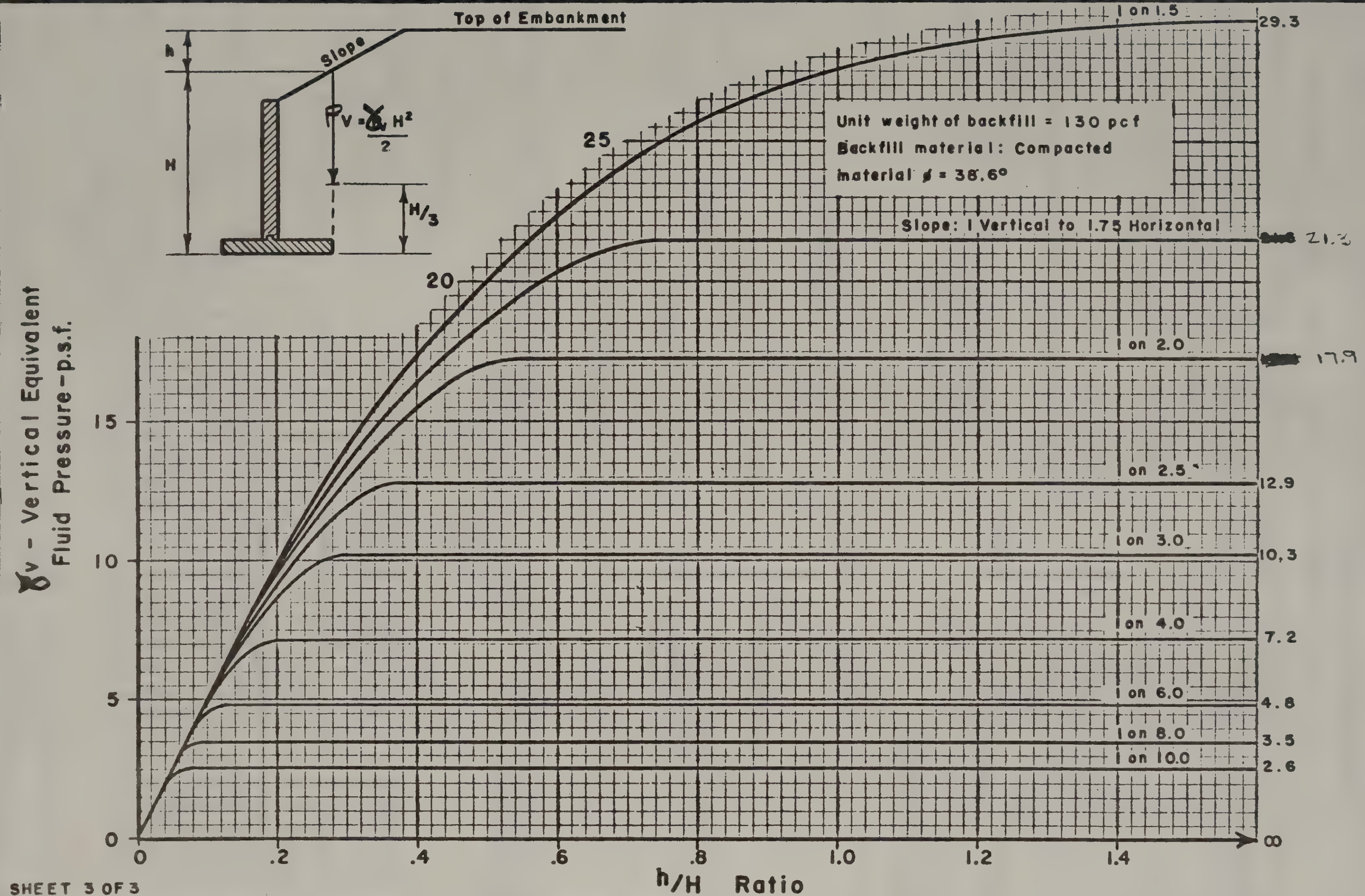

SHEET 3 OF 3

PRESSURES ON WALLS SUPPORTING EMBANKMENTS	EQUIVALENT VERTICAL FLUID PRESSURE	FIGURE 3	STATE OF NEW YORK DEPARTMENT OF PUBLIC WORKS DIVISION OF CONSTRUCTION BUREAU OF SOIL MECHANICS

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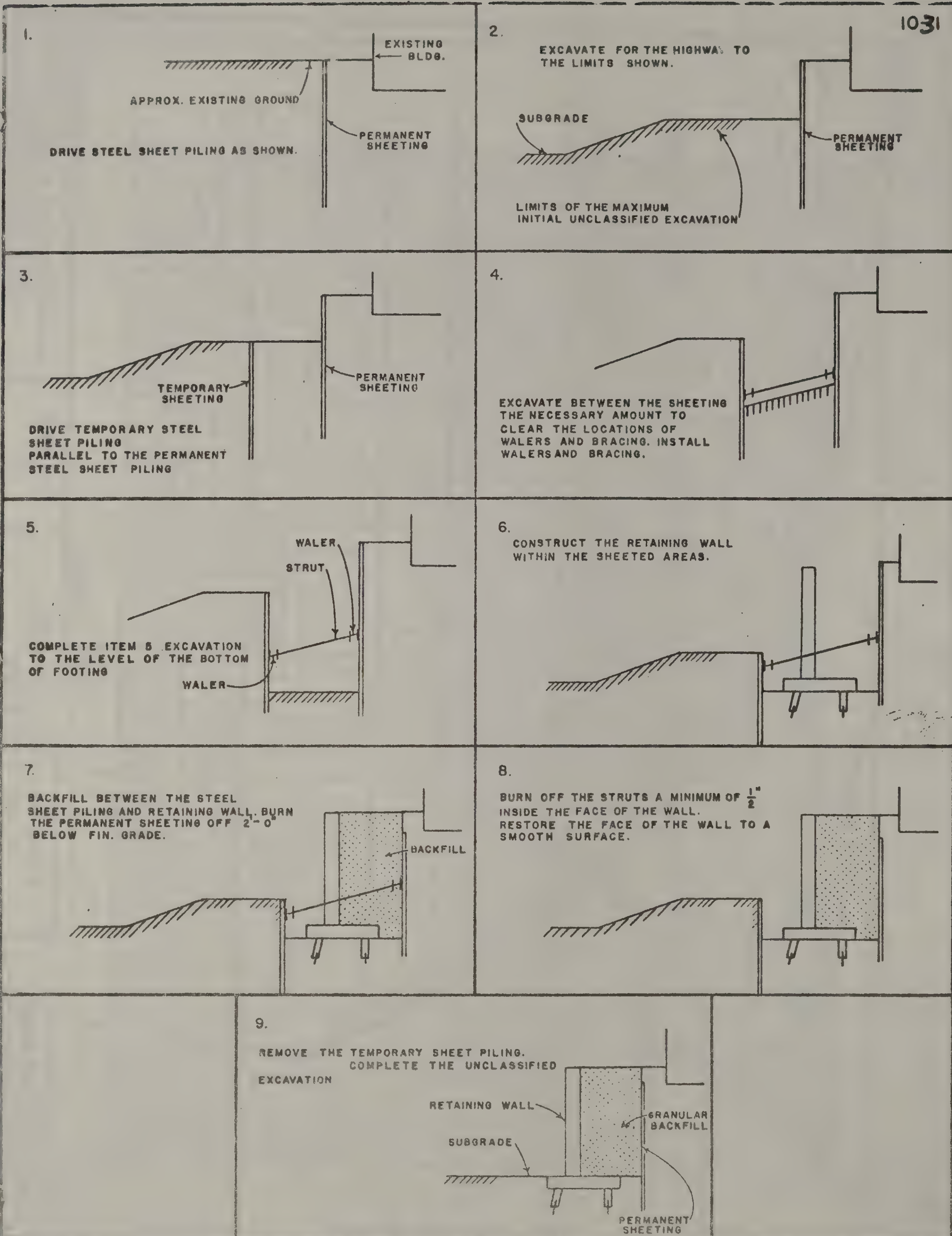
10·30



STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION

SEQUENCE OF CONSTRUCTION OF RETAINING WALLS ADJACENT TO BUILDINGS

Ret. to RSC 10-32

MACCAFERRI GABIONS OF AMERICA, INC.
55 West 42nd Street - New York, N.Y. 10036

GABIONS STRUCTURAL DESIGN DATA — Retaining Walls

Fundamental Facts About Gabions

Gabion walls are essentially gravity structures. Their design follows standard engineering practice, while considering the inherent advantages of their strength, flexibility, and permeability as additional safety factors.

Some advantages of gabion walls are:

1. They may be quickly erected.

2. They are pervious to water and, therefore, afford good drainage.

3. They are flexible. Hence, they need not be founded below the frost line.

4. They are reinforced to tolerate substantial differential settlement without fracture.

5. They are comparatively inexpensive.

Gabion walls are built in courses as in brickwork, for the gabion itself may be likened to a large flexible brick. Walls of the "backward leaning" and "stepped back" types may be built. The choice of type lies with the designer, although the stepped-back type is generally easier to build when the wall is more than about 10 feet high.

The contact pressure on the base of a rigid wall is assumed to be distributed in a planar fashion. Thus, when the resultant of the load on a rigid footing passes through the outer third of the loaded area, the subgrade reaction is assumed to be zero at the heel and a maximum at the toe. The contact pressure on a flexible gabion footing is not distributed in a planar fashion, but decreases from a maximum at the point of application of the resultant to lesser values at the edges of the footing. The pressure at the toe of a gabion wall is, therefore, generally less than for a rigid wall. The calculations for the subgrade reaction on a flexible foundation are awkward, time consuming, and involve errors associated with evaluating the coefficient of subgrade reaction. Therefore, the reaction is assumed to have a planar distribution as in rigid walls as the error in assuming this is on the safe side. See "Soil Mechanics in Engineering Practice" by Terzaghi and Peck for more complete discussion.

Design Criteria for Gabion Walls

The *unit weight* of a filled gabion depends on such factors as the size and shape of the fill material, the method of fill placement, and naturally on the specific gravity of the fill. The engineer should generally specify stone having a size range of 4" to 8" for filling gabions as this size of material has the advantages of being easily handled by mechanical equipment and producing rather small voids when dumped into the empty gabions. However, from measurements taken from a large number of tests using fill of various shapes and sizes, we recommend using a porosity of 0.30 when calculating the unit weight of a filled gabion. The graph in figure 1 may be used to obtain a reliable value of the unit weight for designing gabion walls.

The *coefficient of friction*, f, between the base of a gabion wall and a cohesionless soil may be assumed as the *full value* of the tangent of the angle of internal friction of the soil. This assumption is based on the fact that the surface of a gabion is quite rough and the foundation soil will enter the interstices between the stones in the gabions. Thus, if movement occurs, shear will take place between soil particles, rather than between the soil and gabions. This conclusion may be confirmed by examining the underside of a gabion that has been torn up.

If the wall rests on clay, the resistance against sliding should be based on the full cohesive strength of the clay. If the clay is very stiff or hard, a shallow trench should be excavated where the wall will be built and a 6" layer of well compacted selected clean 3" gravel placed in it.

The *angle of wall friction*, δ, may be assumed as the *full value* of the angle of internal friction of the cohesionless backfill. If no test data are available for the backfill material, a value of 30° should be assumed for the angle of wall friction.

When designing gabion walls to retain clay slopes, a system of gabion counterforts is recommended. These counterforts should be spaced according to table 1.

TABLE 1: SPACING OF GABION COUNTERFORTS*

Type of Soil	Water Content	Cohesion psf	Counterfort Spacing
Very Soft Clay	40%	300	13'
Soft Clay	35%	400	16.5'
Medium Clay	33–30%	600–800	20–23'
Stiff Clay	27–25%	1000–1500	26–30'

**After Reynolds and Protopapadakis*

Gabion counterforts are built as headers and should extend from the front of the wall to a point at least one gabion length beyond the slip circle of the bank.

The counterforts serve as drains and as structural members that support the slope by friction of the bank material against the sides of the counterforts. Hydrostatic pressure in the bank is re-

duced by the free draining material in the gabions. Thus, the thickness of the wall may be reduced because of this combined action of the counterforts.

For a more extended treatment of counterforts, see "Practical Problems in Soil Mechanics" by Reynolds and Protopapadakis. Published by Crosby Lockwood & Son.

For low retaining walls, elaborate theoretical calculations of earth pressure are not justified and semi-empirical methods are generally used instead. A method advanced by Terzaghi and Peck in "Soil Mechanics in Engineering Practice", published by John Wiley and Sons (1948) is shown in figure 2. The values of K_H and K_V are given both in units of psf per lin ft and kilograms per square meter per lineal meter so that walls made of gabions in metric measurements may be quickly calculated.

As with any wall, the foundation should be taken down to a sufficient depth below the circular arc of failure where clay or sandy clay soils are encountered. An example of a gabion toe wall to retain a clay slope is given in "Geology and Engineering, 2nd Edition" by R. F. Legget, published by McGraw Hill (1962).

Figures and Tables

The gabion walls in figures 3 and 4 are proportioned to retain soils of types 1 and 2, as described by Terzaghi and Peck in figure 2. A unit weight of 110 pcf (1760 kg./m^3) has been used for both earth and gabions. If backfill of a different type is used, the wall dimensions do not apply.

The walls were designed so that the resultant is within the middle third of the base, but close to the outside edge as indicated by the arrowhead on each course. The factor of safety against overturning is at least 2.

Of course, safety against overturning is only assured if the pressure on the soil under the toe does not exceed the bearing capacity of the soil. The walls in figures 3 and 4 are safe for soils having a bearing capacity of 2 tons per square foot. Table 2 (Page 4) gives nominal values of bearing capacities for various soils. If the computed pressure exceeds the value for the soil in question, the toe or heel or both must be extended.

The tables accompanying figures 3 and 4 give dimensions for walls built in courses one meter (3' 3") high. Intermediate heights may be obtained by making the foundation course just one-half meter (20") high or by making the top course 12" or 20" high, or both. The foundation course should not be less than 20" high if the stepped back wall type is used as thinner gabions are too flexible to distribute loads effectively.

Because individual gabions encase the stone fill completely, the cross section of the wall may be reduced easily as its height is increased. Also, savings can be effected by eliminating the wire mesh from the hatched portions of intermediate courses as shown in figures 3 and 4. A system of counterforts may be employed in some instances for further economy without sacrificing structural strength. The counterforts are made by arranging gabions alternately as headers and stretchers. For the walls shown in the figures, the spacing of the counterforts is 9' 9" center to center and is obtained by alternating code "A" gabions as headers and stretchers.

While the wire mesh can be partly eliminated in intermediate courses of gabion walls, the entire foundation course must be made up of stone filled gabions to distribute loads effectively.

The stepped back walls in the figures indicate 12" high gabions placed on end at the back of courses 2 and 3 to provide the theoretical base width needed. In practice, it may be easier to use 20" high gabions for course 2 and 3' 3" high gabions for course 3. This will result in a wider wall than necessary, but it may also be more economical to install than the theoretically correct dimensions.

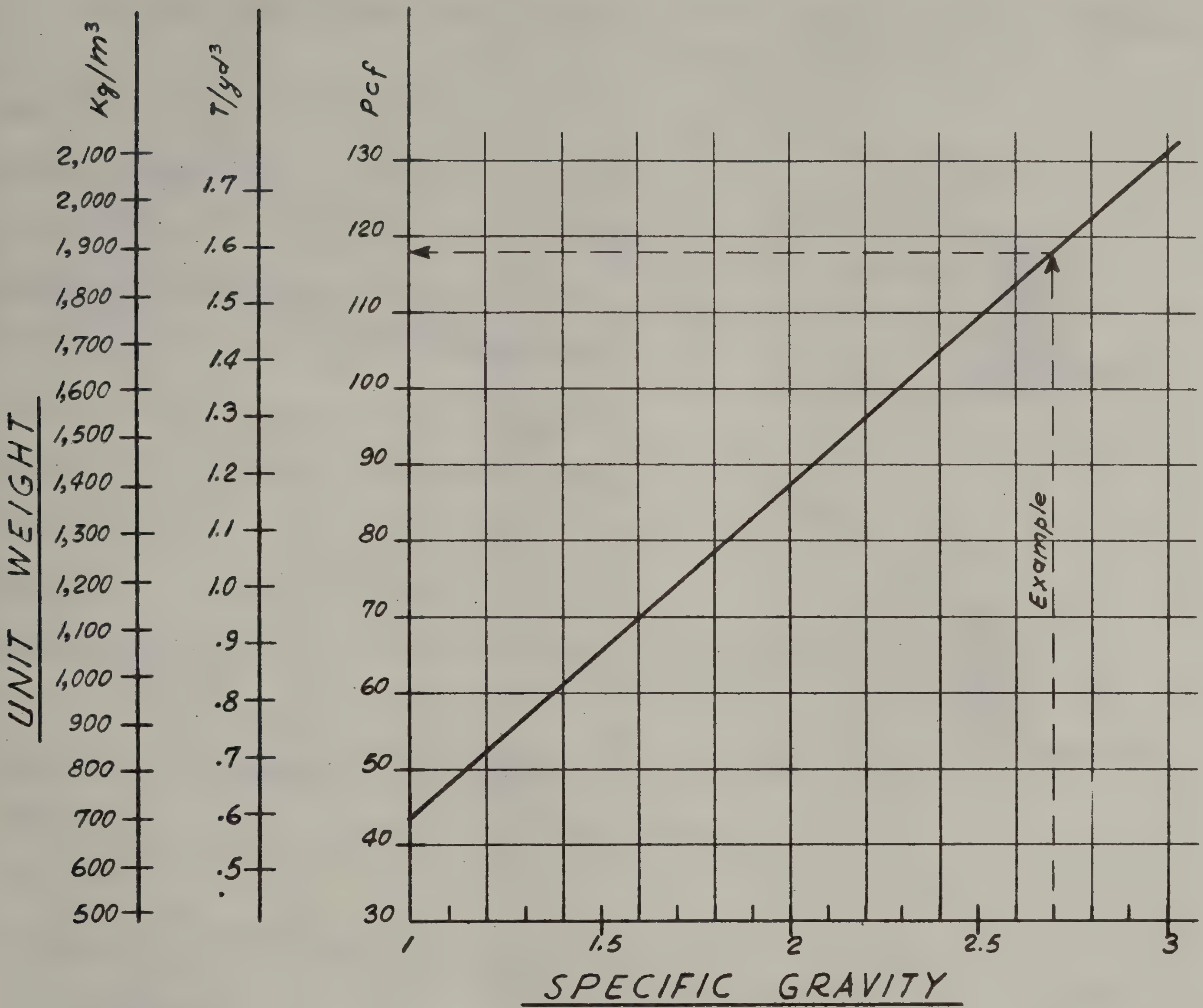
TABLE 2: ALLOWABLE BEARING CAPACITIES OF SOILS*

Type of Bearing Material	Consistency In Place	Recommended Value Of Allowable Bearing Capacity Tons per Square Foot
Well graded mixture of fine and coarse grained soil: glacial till, hardpan, boulder clay (GW – GC, GC, SC)	Very compact	10
Gravel, Gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP)	Very compact Medium to compact Loose	8 6 4
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	4 3 2
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	3 2.5 1.5
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very compact Medium to compact Loose	3 2 1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	4 2 0.5
Inorganic silt, sandy or clayey silt, varved silt – clay – fine sand (ML, MH)	Very stiff to hard Medium to stiff Soft	3 1.5 0.5

*Adapted from Design Manual DM-7, Dept. of the Navy, Bureau of Yards and Docks, Washington 25, D.C.

UNIT WEIGHT OF GABION FILL

(BASED ON A POROSITY OF 0.30)



SPECIFIC GRAVITY OF COMMON MATERIALS	
BASALT	3.0
BRICK	2.0
CONCRETE (BROKEN)	2.4
GRANITE	2.7
LIMESTONE	2.5
SANDSTONE	2.2
TRAP ROCK	2.7

Example:

GIVEN: SPECIFIC GRAVITY = 2.7

FIND: UNIT WEIGHT IN (a) Pcf, (b) T/yd^3, (c) Kg/m^3

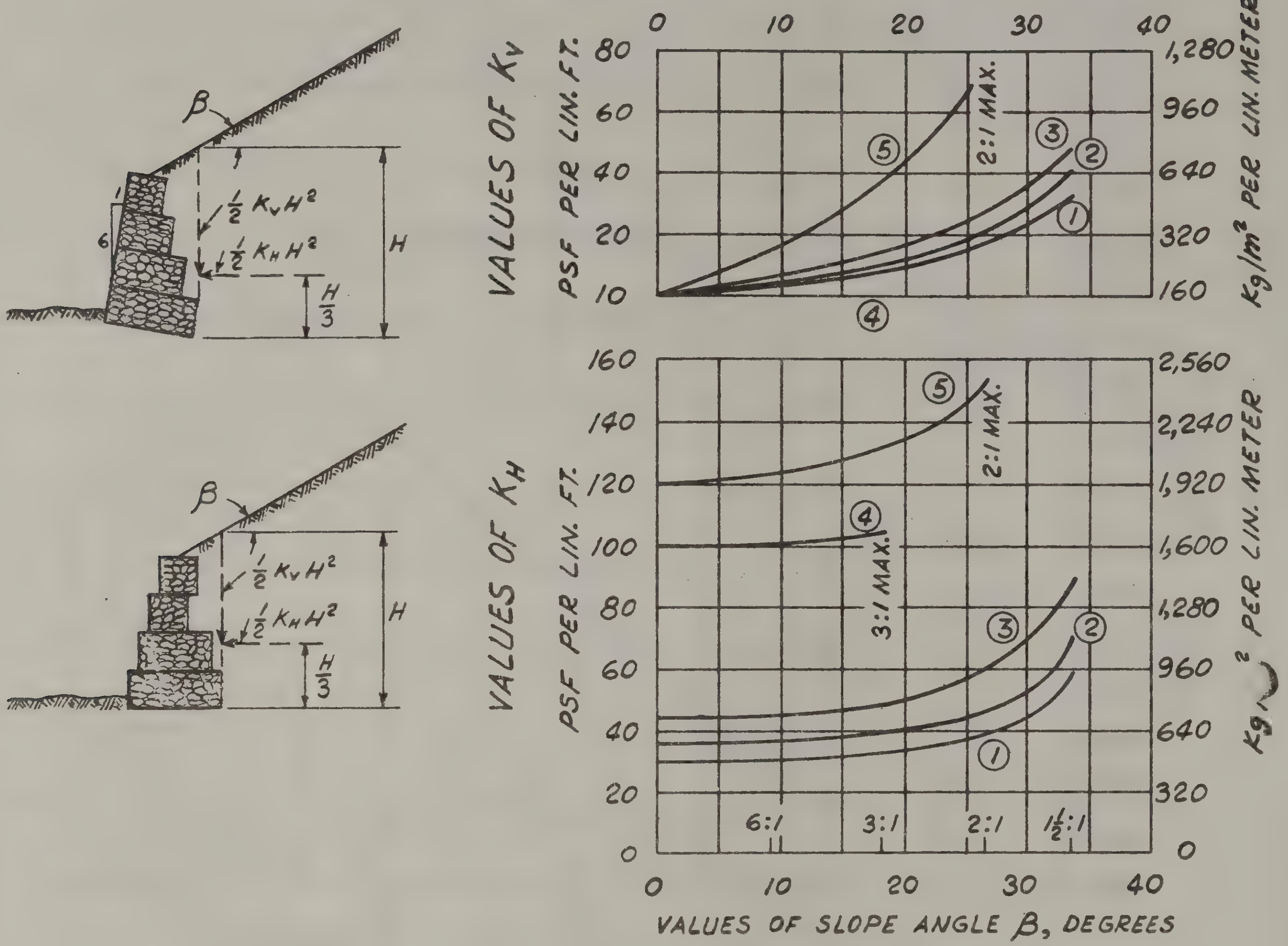
SOLUTION: PROCEED VERTICALLY FROM S.G. = 2.7 TO INTERSECTION OF DIAGONAL LINE. THEN PROCEED HORIZONTALLY TO INTERSECTION OF VERTICAL LINE AND FIND: (a) UNIT WEIGHT = 118 Pcf

(b) " " = 1.59 T/yd^3

(c) " " = 1,890 Kg/m^3

FIG. 1

DESIGN LOADS FOR LOW GABION RETAINING WALLS
(STRAIGHT SLOPE BACKFILL)



CIRCLED NUMBERS INDICATE THE FOLLOWING SOIL TYPES:

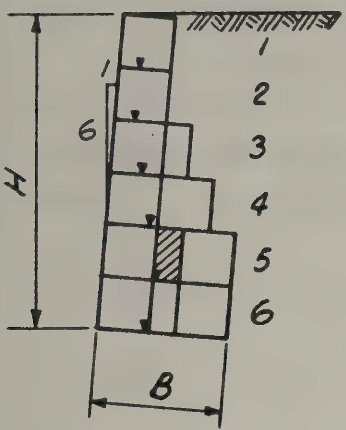
1. CLEAN SAND AND GRAVEL: GW, GP, SW, SP.
2. DIRTY SAND AND GRAVEL OF RESTRICTED PERMEABILITY: GM, GM-GP, SM, SM-SP.
3. STIFF RESIDUAL SILTS AND CLAYS, SILTY FINE SANDS, CLAYEY SANDS AND GRAVELS: CL, ML, CH, MH, SM, SC, GC.
4. VERY SOFT TO SOFT CLAY, SILTY CLAY, ORGANIC SILT AND CLAY: CL, ML, OL, CH, MH, OH.
5. MEDIUM TO STIFF CLAY DEPOSITED IN CHUNKS AND PROTECTED FROM INFILTRATION: CL, CH.

FOR TYPE 5 MATERIAL H IS REDUCED BY 4 FT, RESULTANT ACTS AT A HEIGHT OF (H-4)/3 ABOVE BASE.

FIG. 2

CASE I : $\beta = 0°$

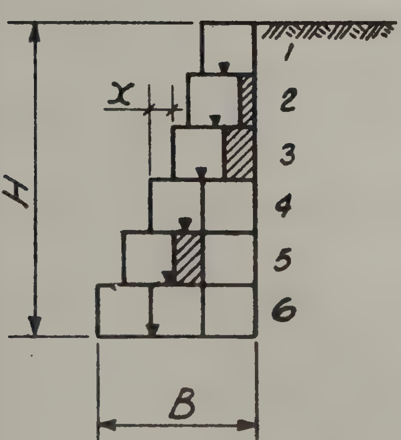
a) FRONT FACE ON 1:6 BATTER



Nº OF COURSES	H	B	REMARKS
1	3'-3"	3'-3"	FRONT FACE MAY BE VERTICAL
2	6'-7"	3'-3"	
3	9'-10"	5'-0"	
4	13'-1"	6'-7"	
5	16'-5"	8'-3"	
6	19'-8"	8'-3"	HATCHED PORTION IN COURSE 5 DOES NOT REQUIRE GABION MESH.

▼ = LOCATION OF RESULTANT OF WEIGHT OF WALL AND EARTH PRESSURE

b) FRONT FACE STEPPED

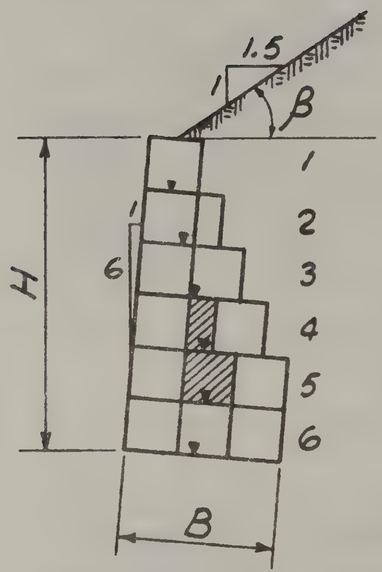


Nº OF COURSES	H	B	X	REMARKS
1	3'-3"	3'-3"		
2	6'-7"	4'-3"	12"	
3	9'-10"	5'-3"	12"	HATCHED PORTION OF COURSE 2 DOES NOT NEED GABION MESH.
4	13'-1"	6'-7"	16"	HATCHED PORTIONS OF COURSES 2 & 3 DO NOT NEED GABION MESH.
5	16'-5"	8'-3"	18"	USE COUNTERFORTS @ 9'-9" IN COURSE 4. SEE NOTE FOR COURSE 4.
6	19'-8"	9'-10"	18"	SEE NOTES FOR COURSES 4 AND 5.

FIG. 3

CASE II: $\beta = 33° 41'$

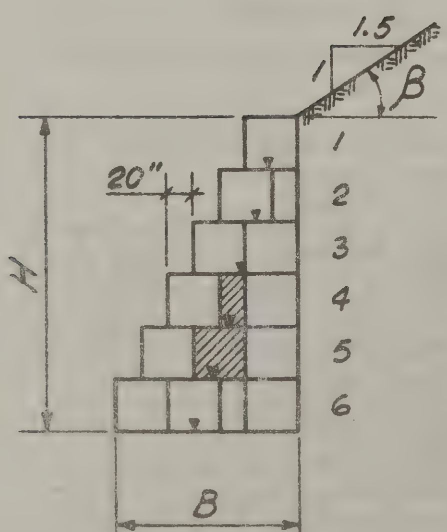
a) FRONT FACE ON 1:6 BATTER



Nº OF COURSES	H	B	REMARKS
1	3'-3"	3'-3"	FRONT FACE MAY BE VERTICAL
2	6'-7"	5'-0"	
3	9'-10"	6'-7"	
4	13'-1"	8'-3"	
5	16'-5"	9'-10"	HATCHED PORTION OF COURSE 4 DOES NOT NEED GABION MESH.
6	19'-8"	9'-10"	HATCHED PORTIONS OF COURSES 4 & 5 DO NOT NEED GABION MESH.

▼ = LOCATION OF RESULTANT OF WEIGHT OF WALL AND EARTH PRESSURE

b) FRONT FACE STEPPED



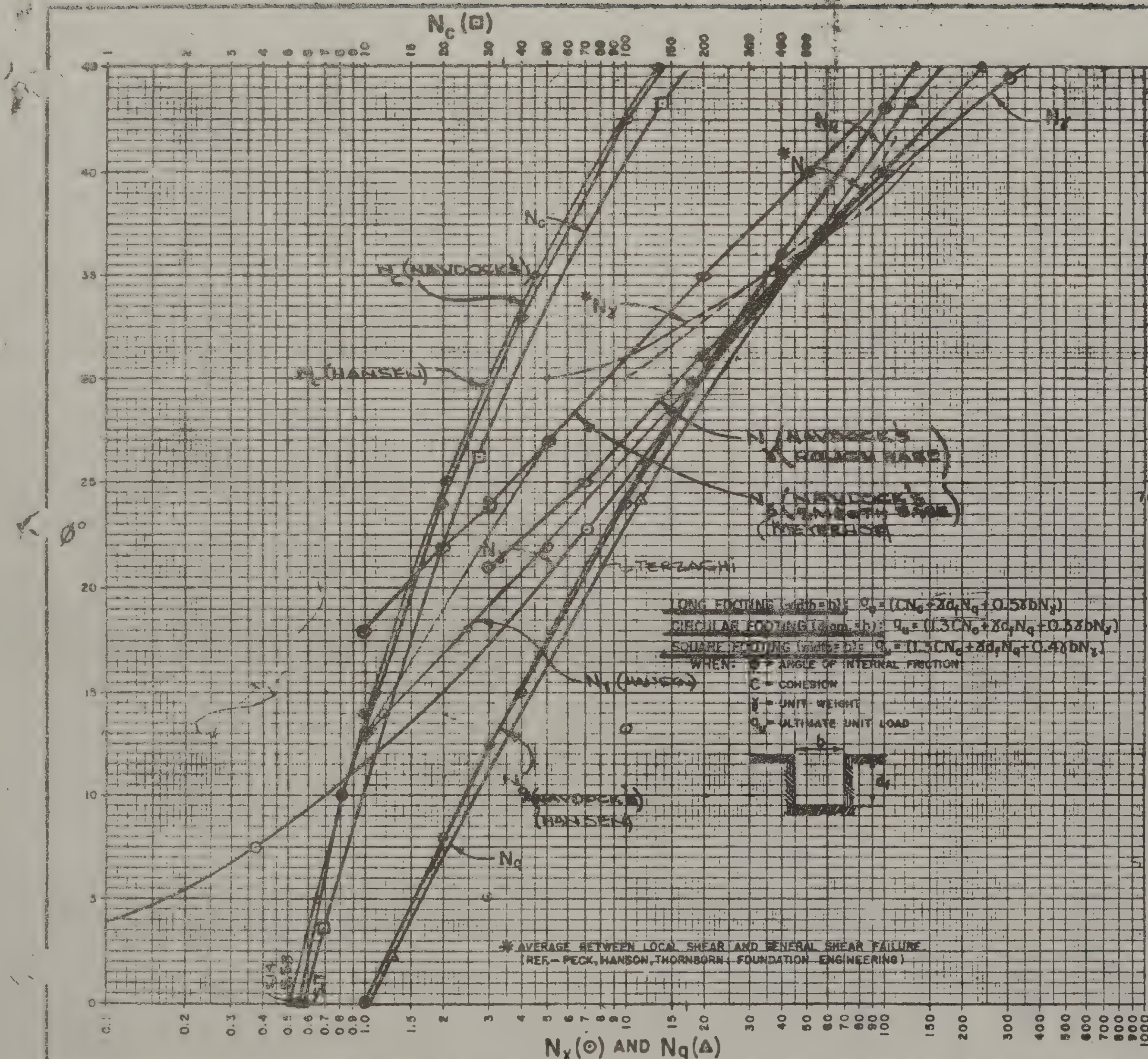
Nº OF COURSES	H	B	REMARKS
1	3'-3"	3'-3"	
2	6'-7"	5'-0"	
3	9'-10"	6'-7"	
4	13'-1"	8'-3"	
5	16'-5"	9'-10"	HATCHED PORTION OF COURSE 4 DOES NOT NEED GABION MESH.
6	19'-8"	11'-6"	HATCHED PORTIONS OF COURSES 4 & 5 DO NOT NEED GABION MESH.

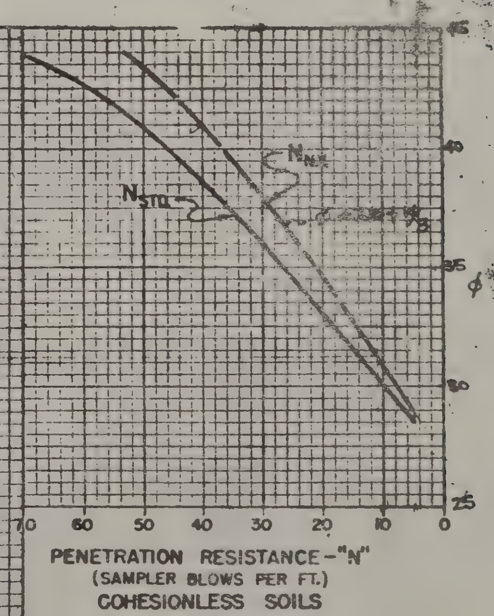
FIG. 4

SECTION 11

SPREAD FOOTINGS

PAGES





NOTES

1) $N_{STD.} = N_{N.Y.}(1.29)$

2) $N_{STD.}$ – SAMPLER BLOWS FOR A 140LB. HAMMER FALLING 30 INCHES.

$N_{N.Y.}$ – SAMPLER BLOWS FOR A 300LB. HAMMER FALLING 18 INCHES.

√3) FOR DENSE, FULLY SATURATED FINE SANDS AND SILTS WITH SAMPLER BLOWS (N) GREATER THAN 15, MODIFY "N" AS FOLLOWS BEFORE OBTAINING N_c, N_γ, AND N_q.

$N' = 15 + 1/2\,(N - 15)$

$D_R > 70$ – GENERAL SHEAR

$D_R < 20$ – LOCAL SHEAR

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

TERZAGHI'S BEARING CAPACITY FACTORS FOR SHALLOW FOOTINGS

GENERAL SHEAR FAILURE

APPROVED 3/17 1950 W. P. Hofmann, PRINCIPAL SOILS ENGINEER	DWG. NO. MO SM 1637

ORGANIZATION OF AMERICAN STATES
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Washington, D.C. May 6-18, 1963

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THE USE OF SOIL MECHANICS IN THE DESIGN AND CONSTRUCTION OF BRIDGE FOUNDATIONS

Philip Keene

United States of America

SUMMARY

Soil mechanics is a rather new branch of civil engineering that is becoming widely used; when applied soundly, it is a valuable aid in solving many or most of the difficult civil engineering problems involving the underground. It is used to solve problems quantitatively, such as amounts of settlement and earth pressures, and qualitatively to correctly visualize and understand phenomena in the underground. On design, the Soils Engineer investigates the bridge foundations problems and gives the Bridge Engineer specific recommendations on them, to make them safe but not wastefully conservative. On construction, the Soils Engineer aids the construction and bridge engineers in features occurring during construction; also, he takes measurements in the field to observe the behavior of foundations and to determine how correct his design recommendations were.

In design, the soils work has three phases: field explorations, laboratory tests, and analyses and recommendations. Field explorations consist of borings, test pits, soundings with bars or pipes, seismic surveys, electrical resistivity surveys, geological studies and other studies such as stream scour. Laboratory tests consist of classification tests to identify the soil and performance tests that measure its performance in compression (consolidation), in shear, in drainage and other functions. Both the explorations and the laboratory testing should be under the direction of the Soils Engineer, as they are in his field of specialized knowledge and the results are used directly by him.

The Soils Engineer then applies the results of the explorations and the testing to his foundation analyses, using soil mechanics theories where necessary, tempered by sound judgment and experience on foundation work. His analyses lead to written recommendations to the Bridge Engineer covering all aspects of the earth and rock as they affect the bridge foundations and substructure. During the above work, the Soils Engineer confers informally with the Bridge Engineer, to learn from the latter the type of structure, number of spans, tolerable settlements and similar features affecting the foundation design. For complete coverage, the Soils Engineer should have a comprehensive check list of all items which he should consider in his analyses and recommendations. This check list includes many items in each of the following categories: (1) elevations of footings, (2) spread footings on earth or rock or both, (3) piles, (4) scour, (5) ground water and river water, (6) effect of roadway fill and (7) miscellaneous. A discussion of some of the more important or less obvious items is given, with examples.

Elevations of footings are influenced by depth of frost, location of a good bearing stratum of earth or rock, safe depth below probable scour and elevation of water table. The last two items are covered in later categories. Where top of a good bearing stratum is not excessively deep, the poorer stratum may be removed and backfill of gravel fill or other low-cost material placed up to bottom of footing. It should be remembered that if the bridge is on piles and the roadway fill is on a slowly-settling foundation soil, a "bump" will result where the pavement meets the bridge floor.

CASSADAGA CREEK

When footings for a bridge are bearing on soil or rock, the analysis of the problem often requires applied soil mechanics. The allowable bearing value is based on the requirement that settlements will be of tolerable magnitudes for the structure in question. If the soil is

sand or gravel or a similar firm stratum or bedrock, suitable bearing values can be assigned from experience, intelligent building codes or reliable text books. The texture of the soil and the blow counts in the borings aid in this. Settlements and lateral movements in such strata are usually small and occur extremely rapidly. If the soil is chiefly silt or clay, a careful analysis should be made; performance tests should be made in the laboratory on "undisturbed" samples and the Soils Engineer makes settlement calculations, using Boussinesq or Westergaard solutions, with modifications if necessary to fit actual conditions. Sometimes we find from the laboratory tests that the clays have been pre-loaded, that is, consolidated during their past history to a load greater than their present overburden load. This means that settlements of the bridge and approach embankment will be much less than if there had been no pre-consolidation. [In central Connecticut, a large number of bridges have been built, without piles, on a rather soft and often very deep varved clay which was preconsolidated. Settlements of these bridges varied from 1" to 8", but differential movements occurred chiefly at expansion joints and construction joints and practically no PERIODIC maintenance work has been necessary. The saving by not using piles was about $2,500,000.]

[If the settlements estimated by the Soils Engineer in the design stage of a bridge are not tolerable to the Bridge Engineer, alternatives must be considered. These are: stage construction, in which the substructure and approach fills are placed and allowed to settle before erecting the superstructure, or pre-loading by placing a fill, with overload, over the entire site and allowing it to settle before starting the bridge, or by using piles, or by changing the type of structure or the bridge site, or by replacing the compressible soil with compacted gravel fill.] The overload-and-wait treatment has been used with great success in recent years in central Connecticut at certain bridges where, in spite of the clay being preconsolidated, excessive settlements of the bridges would otherwise have occurred. [The possibility of ~~a failure~~ STABILITY by sliding should also be kept in mind.] This might occur in clays or silts, but rarely in sands, gravels and hard materials. Analyses made by applied soil mechanics are used in this. Another type of spread footing situation is a stub (perched) abutment on a thick mass of compacted gravel fill. The fill is part of the roadway embankment but is specially constructed to avoid unnecessary settlements.

The subject of piles is both interesting and difficult. Much remains to be known about the behavior of piles and the soil which surrounds them. Our knowledge, based on experience and improved by theories from soil mechanics and other sciences, is usually sufficient to produce proper installations. Types of piles vary and each has advantages and disadvantages, such as compacting or heaving the soil, susceptibility to decay or corrosion, ease of driving and splicing, and cost. Pile driving formulas are commonly used, but are useful only as guides; [static pile load tests are made if an accurate load value is needed. In the interpretation of safe bearing value of friction piles, there are many rules in use.] It appears that the rule in the A.A.S.H.O. book, "Standard Specifications for Highway Bridges", is much too conservative for ordinary cases. [When driving piles in silts and clays, lubrication and remolding of the soil may be serious; [temporary liquefaction of silty soils is also a concern.] Another problem occurs when soil strata settle more than the piles, such as when a recent fill is placed on a soft clay through which piles are driven. The resulting settlement of the fill and clay causes a drag (negative skin friction) on the piles. Research has been made on the problem, as the effect of drag may be large and some costly experiences have occurred because of it.

Scour of stream bed around a bridge foundation does not normally involve soil mechanics, but a knowledge of soil behavior is helpful in studying it. Scour is a very difficult thing to predict and it is extremely important, as it can partially or completely destroy a bridge. Some research has been done on this, including a formula developed in Connecticut after the 1955 floods. Under the category of ground water and river water are matters such as rotting of timber piles, soft conditions at footing excavations, and cofferdams. [Soil mechanics is used in the analysis of a cofferdam, especially if it is to be unwatered; a flow net is drawn and studies made to determine if "boils" will occur and a pervious fill is needed; [then lateral earth and water pressures are calculated and the results given to the Bridge Engineer for his structural study of the cofferdam.]

[In the past, the effect of roadway approach embankments on the bridge abutments was often overlooked, with numerous examples of serious consequences. The roadway embankment causes large lateral pressures in a soft foundation soil, which may result in an abutment on piles being moved horizontally, and it may result in serious drag on piles, as described previously.] [Treatments to avoid or minimize these are by adding an extra span with a stub abutment, or excavating the soft soil, or treating it with sand drainage wells, or by using an overload and a waiting period, or by using struts, or by a combination of these. Soil mechanics is used to analyze these conditions and to make estimates of settlements and lateral movements and pressures.]

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A. Introduction

Like radar, sonar, digital computers and other developments in electronics, soil mechanics is a new branch of engineering that is becoming widely used. Today in the United States it is taught in the engineering schools either at graduate levels or in introductory courses in the senior year. It is accepted by our older practicing engineers as a tool which, when applied soundly, is a valuable aid in solving many, perhaps most, of the baffling and sometimes mysterious civil engineering problems that arise in the underground.

The most obvious use of soil mechanics is in solving specific problems in a quantitative manner, such as the amount of settlement under an embankment or under a bridge, the approximate lateral pressure against a cofferdam, and positive and negative skin friction on piles. However, a more subtle but equally important value of soil mechanics is in the development of the Soil Engineer's understanding and comprehension of many day-by-day phenomena which he encounters in foundation work. Without soil mechanics, the Engineer either does not attempt to interpret many of these phenomena or else he often interprets them incorrectly.

I shall attempt to describe the use of soil mechanics in the design and construction of bridge foundations, based on 24 years' work in this field, including a year of graduate study at Harvard in Soil Mechanics and Foundations Engineering, 23 years as the chief Soils and Foundations Engineer for the Connecticut Highway Department, and study of the current literature, and discussions and conferences at conventions and meetings such as this one.

In bridge foundations the Soils Engineer can do a great deal to aid in securing proper design and construction of these foundations. The task is primarily an engineering one. For that reason the soils engineering organization should be an engineering unit rather than a laboratory unit or a geological unit. On bridge foundation work during design, the soils engineers should work closely with the bridge engineers, the Soils Engineer giving the Bridge Engineer specific recommendations regarding foundations for the structure in question. These recommendations can be used by the Bridge Engineer to make a rational foundation design, which is safe and yet not wastefully expensive. When aspects of the Soils Engineer's recommendations are questioned by the Bridge Engineer, or when the type or location of the bridge or certain details are changed, the Soils Engineer and the Bridge Engineer should be in close touch to straighten out the matter promptly and rationally. Speaking from my own experience, the effectiveness of soils engineering was doubled or tripled, both regarding bridge design and highway design, when the soils engineers were transferred from the Highway Testing Laboratory to the Engineering and Construction Bureau at Hartford 17 years ago.

Similarly, during construction, the Soils Engineer works with the construction forces, including bridge engineers, giving them recommendations based on his specialized knowledge. Furthermore, in soils work it is very important to verify his recommendations during design with field observations during construction. "The proof of the pudding is in the eating." My soils engineers visit construction projects, whether requested or not, to learn how accurate their recommendations proved to be, and we record, in a special book, references to projects having special features, unusually accurate field observations, etc. for use on future projects. Today, because our specialized work is used on both design and construction, and in connection with bridges, roadways, highway buildings, rights-of-way and research, our Soils and Foundations Division is in a staff status, under the Assistant Chief Engineer.

For bridge foundations in design, the soils work can be divided into 3 phases: (1) field explorations, (2) laboratory tests, and (3) FINAL analyses and design recommendations.

B. Field Explorations

PRELIMINARY ANAL & TEST REQUEST

These consist of test borings and other subsurface explorations where desirable. The latter include pits, soundings to shallow bedrock or to locate a suspected rock fill, seismic or electrical resistivity, examination of outcrops at ground surface and in basements of buildings, study of geologic maps, evidence of scour, effect of new construction on scouring characteristics of stream, and type and performance of existing foundations. Knowledge of geology is, of course, necessary in most of this and professional geologists are a desirable part of the soils engineering staff.

I shall not take space here to describe boring operations or other subsurface explorations, as they are highly specialized work. I should say, however, that this work should be under the Soils Engineer as he is the one who has the primary use for borings and other explorations; he understands soil and rock behavior and can interpret with the drill crews certain phenomena which might be baffling, e.g., loss of wash water, no recovery or small recovery

of sample. Puzzling laboratory test results often can be explained by the Soils Engineer through his knowledge of the conditions during the boring and sampling operations. Also, the Soils Engineer is the one who decides on the number of holes, necessary depths, necessary samples, etc.

C. Laboratory Tests

There are a number of soil tests, many of them quite well standardized. They are divided into classification tests (to identify the soil), such as grain-size analysis, water content, and liquid and plastic limit, and performance tests, such as consolidation (settlement), triaxial shear, direct shear, vane shear, unconfined compression and permeability. These should be under the supervision of the Soils Engineer for the same reasons as for the subsurface explorations. The testing should be done, preferably at or close to his office. This will enable him to watch special tests and note special features which may be overlooked by the soils testing man who usually is a technician rather than an engineer. That situation occurs fairly often. Also, if a test result appears peculiar, the Soils Engineer goes to the laboratory to attempt to learn the reason for the apparent peculiarity. Partial disturbance of an undisturbed sample, partial drying or wetting during the test and occasional thin layers of a different soil type are some of the things which require the special attention of the Soils Engineer. Therefore, the closer the laboratory is to the Soils Engineer's office, the more convenient and more effective is his supervision.

D. Analyses and Recommendations for Design

The Soils Engineer who is assigned to a certain project should make tentative analyses and recommendations, written or verbal, to the Bridge Engineer who is designing the bridges on that project as the first two stages (borings and tests) are proceeding. This allows the scope of those stages to be altered, if necessary, while they are still in progress. When those stages are completed, the Soils Engineer can complete his analyses and make his recommendations in writing to the Bridge Engineer.

In making his analyses he confers informally with the Bridge Engineer over details such as final location of piers, abutments and wing walls, type of superstructure and permissible total and differential settlements. The Soils Engineer's analyses are based, where applicable, on theoretical and applied soil mechanics, tempered by sound judgment and experience. His experience should include knowledge on construction of foundations, preferably with field measurements, such as settlements, horizontal movements, and pile behavior. Frequently past experience and knowledge of geology are used for certain problems. His recommendations for design should cover all aspects of the earth and rock as they affect the bridge foundations and substructure. To insure that none of these aspects may be overlooked, a check list, such as the following, is useful:

1. Elevations (or depths) of bottom of footings, including wingwalls.
    a. Frost
    b. Scour
    c. Remember to core rock deeper if footings will rest on it.
    d. Watch for possible muck not found by borings.
    e. "Steps" in bottom of footing on $1\frac{1}{2}$:1 slope for soil and $\frac{1}{2}$:1 for rock.

2. "Spread" footings bearing on soil or rock or both
    a. Allowable (design) unit loads, average and maximum, under footings.
    b. Estimated settlements.
    c. Desirability of stage construction and overload.
    d. Vertical expansion joints.
    e. Stability against sliding.
    f. Special fill under perched abutments.
    g. Soil load tests.

3. Piles
    a. Type
        (1) Rotting above future ground water table.
        (2) Marine borers
        (3) Corrosion
        (4) Displacement and vibration of soil - compacts or heaves
        (5) Deep penetration
        (6) Subsurface obstructions
        (7) Cost

b. Methods
   (1) Leads
   (2) Followers
   (3) Hammers
   (4) Sequence of driving
   (5) Jetting
   (6) Interference with adjacent piles and cofferdams

c. Bearing Values
   (1) Formula - rough guide
   (2) Load test - for friction piles
   (3) Liquefaction in clays and silts
   (4) Skin friction values for various strata
   (5) Negative skin friction in settling strata
   (6) Resistance of soil to lateral movement of piles

d. Estimating Lengths
   (1) Embedment in footing
   (2) Embedment in soil
   (3) Embedment in rock
   (4) Additional for batter lengths
   (5) Buckling or brooming at top due to driving
   (6) Irregularities of strata
   (7) Extra to avoid splicing
   (8) Minimum penetration because of scour danger

e. Miscellaneous
   (1) Timber piles, end-bearing - not over 25 tons
   (2) Timber piles, - steel shoes in special cases
   (3) Steel shells - thick, if driving through bouldery fill
   (4) Steel shells - reinforcing in upper part
   (5) Pile cap if footing concrete will settle during hardening

4. Scour
   a. History from local people
   b. Performance of existing structures
   c. Scour formula for small bridges
   d. Sheeting
   e. Rip-rap
   f. Slope paving
   g. Stream-lining ends of piers

5. Ground Water and River Water
   a. Possible future Ground Water Table
   b. Effect of Ground Water Table during construction
   c. Type and stability of cofferdams
      (1) Water pressures by flow net
      (2) Earth pressures
      (3) Gravel fill
      (4) Concrete pile cap
      (5) Tremie seal
   d. Well points

6. Effect of Roadway Fill behind Abutments
   a. Settlements of footings (see Item #2b)
   b. Settlements of piles (see Item #3c-5)
   c. Lateral earth pressures
   d. Type of foundation soil under fill
   e. Stability against sliding
   f. Lateral movement of piles and/or walls (see also Item #3c-6)
   g. Weep holes or underdrains
   h. Pervious fill

7. Miscellaneous
   a. Piers to be installed before stub abutments
   b. Disposal of bridge excavation material

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A discussion of some of the more important items and some of the less obvious ones is desirable, illustrating with examples.

Group #1 - Elevation of Bottom of Footings. These elevations should be established by the Soils Engineer from one or more considerations: depth of frost, location of a good stratum such as gravel or sand or bedrock, safe depth below probable scour, and elevation of ground water table. The last two will be discussed later. It sometimes happens that a footing can be kept high, in a gravel stratum which overlies a more compressible soil, and thus avoid the need for piles.

3x

Sometimes at a compressible stratum a footing can be lowered to within about 2 ft. of the bottom of that stratum and placed on about 2 ft. of gravel fill to avoid the need for piles. A case occurred in Connecticut a few years ago at an expressway grade separation bridge. There, 7 ft. of soft alluvial silt was underlain by 21 ft. of medium sand and then 10 ft. of clay to bedrock. The silt was removed and 2 ft. of compacted gravel fill placed to support the footing. Settlements were about 2". The net saving in foundation costs was about $25,000. An advantageous by-product of omitting the piles where the foundation soil is compressible, is that the bridge settles with the roadway fill, which aids in reducing the "bump" where the pavement meets the bridge deck.

A minor but sometimes troublesome item is the step in a stepped footing. A vertical step in earth is difficult to cut and rarely will stand without cracking or sloughing, especially when workmen are around. Once it is disturbed, it is of no value in supporting the footing unless thoroughly re-compacted. The realistic design is to have the step rising on about a 1-1/2 to 1 or 2 to 1 slope. Similarly, a step in rock should be on about a 1/2 to 1 slope for most conditions.

An unusual case illustrating the need to core rock deeply where a footing will rest on it occurred many years ago in Connecticut. Borings for an arch bridge in a small gorge encountered bedrock at shallow depths, so footings on bedrock were recommended. It was discovered during construction that "bedrock" at one boring was a large boulder, about 6 ft. thick. Apparently the boring went into but not through the boulder. If the boring had been deeper, it would have gone through the boulder and the true bedrock surface would have been found.

Group #2 - Footings Bearing on Soil or Rock or Both. The analysis of the problem of a footing bearing on soil can generally be made by using applied soil mechanics. The allowable bearing value is based on the requirement that the settlements shall be of a tolerable magnitude for the structure in question. Occasionally the requirement that the footing shall not fail by rupturing (SHEAR) the soil is also investigated, but if the soil is so soft in shearing strength that rupturing (sliding) might occur, it is usually so soft that settlements would be excessive and intolerable. If the soil strata are sand or gravel, or compact well-graded mixtures such as glacial till, suitable bearing values can usually be assigned from past experience, intelligent building codes or reliable text books. The texture (gradation) of the strata and the blow counts in the boring report usually provide sufficient data for proper evaluation of such soils. Settlements and lateral movements of these soils are nearly always insignificant and occur almost instantaneously as each load increment (footing, wall, superstructure and backfill) is applied.

If the soil strata are chiefly or wholly clays or silts, a careful analysis should be made by the Soils Engineer. These soils are usually more compressible than sands, gravels and tills, but fortunately they can be sampled without great difficulty, so that good drive samples and so-called "undisturbed" samples can be obtained in clays and silts. During or after the boring and laboratory test work, the Soils Engineer makes settlement calculations for abutments and piers. He uses tentative allowable footing pressures and tentative dimensions of footings, and secures the data on probable roadway fills and cuts. From these, he calculates the future stresses in the underground and compares them with the existing stresses, using the Boussinesq or Westergaard solutions, with modifications if necessary to fit the actual conditions. In these calculations, the negative loads caused by excavation for footings and roadway cuts, the positive loads caused by roadway fills and the buoyancy due to present and future ground water must be included. It can be seen from Fig. 1 that, unless the compressible stratum under the footing is shallow, the weight of the roadway embankment contributes much more than the weight of the bridge to settlement of the bridge. BEC PRESS. BULB OF EMB. = ~~GREATER~~ DEEPER THAN FOR FOOTING

P_{on}

The vertical deformations (settlements) due to the increases in vertical compressive stress are calculated from laboratory consolidation tests on representative undisturbed samples. We frequently find clays that have been pre-loaded, that is consolidated during

$S = \frac{H \Delta e}{\ldots} = \frac{H C_c \log P_F/P_o}{\ldots}$ & C_R IF $P_F < P_P$ $= \underline{H \log P_F/P_o}$

their past history to a load which is greater than their present over-burden load. This condition, which is revealed by the laboratory consolidation tests, (see Fig. 2 and 3), is very beneficial. The reason is that if a load is placed on a soil, then part of the load is removed, and then the load is reapplied, the consolidation under the reapplication of load is much less than under the original application of load. It means that future load increases due to the bridge and embankment will cause relatively small settlements, for some or all of these new loads will be merely restoring the ancient load on the soil. In the north-central part of Connecticut are extensive areas containing thick deposits of gray and red varved clays, extending to a maximum depth of about 275 ft. below ground surface. These were deposited in what geologists call Glacial Lake Connecticut or Hitchcock. These varved clays have been preconsolidated to loads varying from one to four tons/sq. ft. greater than their present over-burden load. Hence, we have successfully built numerous grade separation bridges on them without using piles. These bridges settled uniformly, some 1" or less and others as much as 6" depending on thickness of clay and other factors. In 1959 at the suggestion of the U.S.B.P.R., I made a survey of these bridges, 22 in number, and furnished them with copies of the data sheets. The survey revealed that at 14 bridges, actual settlements differed from the design estimate by 1/2" or less and at the other 8 bridges the difference was about 1". In one-half of the cases, the estimate was too low and in one-half it was too high. The survey also revealed that most of the differential movements in the structures occurred at expansion joints and construction joints; elsewhere, there were some vertical hairline cracks in abutment walls. Also I found no maintenance work had been considered necessary at these movements, except pointing up a few joints with fresh mastic. The magnitude and frequency of these differential movements ("defects") are about the same as for our non-settling bridges on piles or on spread-footings. I might add that one of the bridges in the survey is a two-span continuous rigid-frame steel structure, with pier on piles to hardpan but with abutments resting on clay. The abutment piles were eliminated during construction in 1941, at the Contractor's request, due to the shortage of steel. The abutments settled about 1-1/2", slightly more than I estimated, but no harm has resulted. The total saving on these 22 bridges, by not using piles, was about $2,500,000.

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Obviously, where reliable settlement and soil data at similar locations in the area are available, these should be used to check or modify the above settlement predictions. Calculated amounts of settlement commonly have an error of 20%, plus or minus, and calculated rates of settlement are less accurate, because of the strong effect of thin lenses of sand. Hence, settlement observations of structures or embankments are an important aid.

With his estimates of amounts and rates of settlement, the Soils Engineer confers with the Bridge Engineer to determine if such settlements are tolerable in the proposed structure. If they are not, there are several alternatives: stage construction, by which the substructure and approach fills are placed and allowed to settle before erection of the super-structure, or by pre-loading by placing a fill with overload over the entire site and allowing it to settle, or by the use of piles, or by a change in the type of structure or a change in location, or by replacing the compressible soil with compacted gravel fill.

3 MO.'s GOOD FIGURE IF FEASIBLE

Stage construction with a settlement period of one to six months is used occasionally in Connecticut; it causes no great inconvenience or delay if the bridge is part of a large project, as the contractor can schedule his work to fit the requirements. Perhaps the most interesting and unusual example in Connecticut of the overload-and-wait treatment is on three grade separation bridges on the East Hartford Expressway, where the varved clay is 80 to 100 feet thick. At each bridge, the 20 ft. fill for the underpass and the 40 ft. fill for the overpass were both placed as though no structure were to be built. After the specified six-months' wait, the overpass fill was removed sufficiently to permit building the bridge. Settlement at the time this overload was removed amounted to about 15"; our readings on our numerous settlement plates and piezometers during the last three years indicate that one bridge will settle about 8" and the other bridges will settle about 4". On this same project is a 660 ft. box culvert which has settled about 18" maximum to date and will settle about 3" more, with no harmful effects observed except small openings of the expansion joints. The saving by not using piles on the 3 bridges and the box culvert is nearly $500,000.; also, there is no bump where the pavement of the bridge approach meets the bridge deck, as the embankment and bridge are settling together. The success of this treatment has led us to use this method on several other bridges now under construction near there.

The alternate of a change in type of structure sometimes occurs. An example of this is in changing the structure from a continuous type to one having simple spans; this will increase the cost slightly but may be desirable if intolerable differential settlements are anticipated.

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Where differential settlements are anticipated in an abutment and wingwalls due to conditions like the above or due to the footing being partly on soil and partly on rock, the Soils Engineer can aid the Bridge Engineer in selecting advantageous locations for vertical expansion joints. Such joints usually "take up" differential movements in the substructure and, thereby, eliminate cracks in the latter.

Before leaving the subject of allowable bearing values under a footing, a few remarks should be made regarding the criterion that the bearing on the soil should not cause failure due to a rupture of the soil. This criterion was mentioned previously, but not discussed. Rupture of the soil beneath the footing due to an excessively large load on the footing would occur, with customary highway structures, only in clays and silts. As the footings are placed below ground, a failure in sands, gravels, and compact strata such as tills would occur only under rare conditions. In clays, ultimate bearing value of the footing is approximately 5.5 times the shearing resistance of the clay plus the weight of the soil above the elevation of bottom of footing. Consequently, if a footing is founded at 5 ft. below ground surface on a clay whose shearing resistance is about 600 psf. to at least 10 ft. below footing and whose density is 110 pcf., the ultimate bearing value of the footing is about 5.5x600 plus 5 ft. @ 110 = 3850 psf. The determination of the shearing resistance of the clay is difficult: the unconfined compression test is believed to give less than the true value and the field vane shear test and the triaxial compression test seem more reliable; the entire subject of shearing strength of clays is complex and is now the subject of much research and a considerable number of seminars and conferences in this country and abroad.

5.7 C

A word also should be said regarding a chart of bearing values given in "Soil Mechanics in Engineering Practice", by Terzaghi and Peck.* This chart, Fig. 177 on page 423, gives approximate allowable values, based on the "standard" penetration test when obtaining drive samples of soil in test borings. These allowable values in tons/sq. ft. are equal numerically to approximately one-tenth of the number of blows required to drive the sampler pipe one foot; if ground water table is close to the footing, the allowable value is reduced. Our experience is that these values are much too conservative for highway structures. For buildings, these values may be more proper, as footings may be at ground surface, the structure is more delicate, and there is no heavy substructure and backfill to cause the major settlement before the superstructure is erected. Furthermore, the authors give words of advice on page 413, stating that preference should be given to past experience and local practice, which should be followed, rather than the chart on page 423. Included on page 413 is the following: "The accumulation of the observational data needed for establishing local rules is a very good investment and should be encouraged."

.15 → .2 IN WAY DOCKS

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A final item which can be discussed is compacted fill, usually several feet thick, under perched (stub) abutments. Such fill is part of the roadway embankment, but it should be specially compacted and should generally be of special material. In Connecticut, we have considerable clean bank-run gravel and frequent rainfall. Consequently, we use the gravel for this special fill, as our natural soils frequently have much silt and sometimes clay, and become spongy if placed during prolonged wet weather. On the other hand, the gravel compaction is improved by rainfall. The gravel is placed in 8" layers, compacted to 100% of Modified Proctor (laboratory) density. The latter is performed in accordance with A.A.S.H.O. Test T-180, Method D. We usually require a soil load test on a 6' x 6' area when the fill is completed, as a check on the adequacy of the work. These tests usually show a gross settlement of less than 1/2" at twice design load. This special gravel fill costs more than ordinary embankment but is much cheaper than using piles, unless the gravel fill is unusually high, such as 20 ft., and the piles would be short, such as 25 ft. However, when scour of the foundations is possible, it would be foolish to use the fill instead of piles.

Group #3 - Piles.

The subject of piles is both interesting and difficult. Much remains to be known about the behavior of piles and the soil which surrounds them, particularly in the case of friction piles. Papers have been written describing notable installations and careful measurements taken, but in some cases the explanation of the behavior of the piles in relation to the soil could not be readily explained. However, for the usual project involving piles, our knowledge, based on experience and improved by theories from soil mechanics, is sufficient to produce safe and reasonably economical installations.

Types of piles for most highway bridges are currently of three main types: timber, cast-in-place concrete, and steel H-piles. However, pre-cast concrete piles are used commonly

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*"Soil Mechanics in Engineering Practice", by Karl Terzaghi & Ralph Peck. John Wiley & Sons, (1948).

in some areas and prestressed pre-cast concrete piles are gradually becoming more widely used. Timber piles are generally the cheapest to furnish and will displace and compact a loose pervious soil, such as a loose sand; however, they will rot unless creosoted or otherwise treated, when above ground water table, are vulnerable to marine borers and should not be loaded to high unit loads when they are end-bearing. Cast-in-place concrete piles will compact loose pervious soils and can be easily extended in length by splicing. H-piles can be extended and can be driven to great depths if necessary and between subsurface obstructions in certain cases; they will cause a minimum of heaving when penetrating impervious soils such as clay, due to their relatively small displacement volume, but for the same reason, are only moderately effective in compacting loose pervious soils.

Timber or concrete piles which have an average diameter of 12" and an average spacing of 3-1/2 ft. will displace a volume equal to about 6% of the soil into which they are driven. On the other hand, 12" H-piles will displace a volume of only about 1%. Hence, when a timber or concrete pile, 60 ft. long, is driven into a deep stratum of sand, and causes neither subsidence nor heave of the ground surface, it is compacting the sand about 6%. When a 12" H-pile of that length, however, causes a subsidence of about 12" in such material, which is not unusual, this means a compaction of only about 2-1/2%.

Bearing values of piles are undoubtedly the most controversial phase of the subject. End-bearing piles usually give no trouble in this matter, as virtual refusal is generally attained when driving them. Friction piles, however, are often given bearing values by a variety of methods. Various formulas based on the blow count during pile driving, energy of the blow, and other factors are the commonest method of determining allowable bearing values. The best known is the Engineering News formula. This formula usually gives a factor of safety, when compared to total failure by plunging, between 3 and 5 when the soil is sand or gravel. However, when the soil is silt or clay, the formula is of little or no value when driving continuously, due to the lubrication which develops along the surface of the pile. The pile disturbs and displaces the soil adjacent to it; since silts and clays have low permeability, pore water does not have sufficient time to be squeezed out of this soil and thereby permit the soil to densify. Hence, the soil retains its natural water content and is merely softened by remolding next to the pile. A better estimate of allowable bearing value is to use the blow count after a long rest of several hours, such as overnight. However, the best method of determining bearing values of friction piles is by a static load test, usually carried to twice the design load. While a test on a single pile is usually satisfactory for a routine project, a group test is necessary when soil conditions are bad. A case in point is a bridge site in Connecticut having below stream bed 165 ft. of soft pink silt underlain by 100 ft. of fine sand and silt. A load test on a group of four 12" concrete piles, 70 ft. long, was specified, as well as the installation of the 11 piles adjacent to the test piles. By loading 4 piles, the detrimental effect on a loaded pile due to the load on a neighboring pile could be included, as well as the deep settlement due to load from four instead of one pile. The 11 adjacent piles were driven to create the benefit or detriment to the soil at the group caused by installing adjacent piles. Actually, the Contractor drove 15 additional piles to serve as anchors for the "bootstrap" load test, making a total of 30 piles driven. The piles drove very easily. Ten days after the piles were driven, the load test on the group of 4 piles was begun. Increments were added every 48 hours, and on application of the third increment, making a total load of 44 tons/pile, they had settled 13". But when 30 ft. extensions were added two days later, the piles were driven with great difficulty. Then a second load test was made on the four piles, two weeks after the first test, and the piles settled only 3/4" at 70 tons/pile. The balance of the bridge piles were made 90 ft. long. Each abutment settled 3/8" and moved about 3-1/2" toward the river. Provision had been made in the design for these movements.

Another case of temporary liquefaction of fine-grained soil occurred in 1956 at a bridge abutment where 30 ft. concrete piles were driven into a fine sand, containing about 25% silt, which extended to at least 60 ft. below the footing. Load tests on isolated test piles gave less than 1/4" settlement at design load. However, the regular production piles drove so easily that a load test was made on a "soft" pile three weeks after all were driven at this abutment, although a little driving was in progress at the other abutment. The load test showed a 1" settlement at design load, and it plunged during the next load increment. Finally, another load test was made 8 days later, when all piles for the bridge had been driven; settlement at twice design load was only 3/8". Observed settlement of the abutment, with embankment in place, is less than 1/4".

Another difficult problem connected with piles arises when one or more settling strata exist at the structure. A frequent example is a recent fill placed on a thick layer of soft clay or mud. As the strata settle, they tend to drag down the structure by negative skin friction. This drag can be roughly calculated by assuming active lateral pressure of

cohesionless soil against the piles and structure, and by using the actual shearing strength in the case of cohesive soils such as clays. There is little doubt of the presence of this drag. At New Haven Harbor, Connecticut, there are large areas of thick sand fill laid on a thick layer of marine mud. One could see cracks in the fill above culvert structures supported on piles; these cracks were located directly above the outer edges of the structures. These structures, on deep piles, have settled only about 1", even though the subsequent adjacent fill has settled several feet. In contrast, an isolated experimental pile was driven, to a high formula value, through the fill and mud and penetrated the fine sand below for a few inches; the pile settled 9", with the fill, in one year. A similar case occurred a few years ago near Boston, where a large warehouse was built on piles driven through gravel fill overlying peat. At one corner of the building, the piles barely penetrated through the peat and consequently settled 7", which resulted in costly underpinning to repair the damage. In an attempt to evaluate this drag, or negative skin friction, on piles we made an extensive research project as part of a bridge contract in New Haven in 1955, using SR-4 strain gauges placed at various depths inside 3 test piles.* The results were very useful, as a guide for future design.

Estimating pile lengths is done by the Soils Engineer as part of his recommendations, using his knowledge of the geology of the site, the values of the soil and rock and pile-driving practice. In Connecticut he makes recommendations during construction to the Construction Engineer for pile order lengths to be given to the contractor. This recommendation is based on test piles driven by the contractor, load tests (if any) and his background of investigational work made during design.

Before leaving the subject of piles, there are two things which should be mentioned regarding the pile specifications in the well-known A.A.S.H.O. book, "Standard Specifications for Highway Bridges"** I am familiar only with those parts of the book that deal with soils and foundations, but I judge it is an excellent manual for bridge design and construction. However, its specification on interpretation of safe bearing value of piles from load tests, Article 2.3.6(a), is very old and should be overhauled. This specification has been unchanged, except for minor phrases, since the first edition of the book in 1931, which is 32 years ago. It states that, when the net settlement (after all load has been removed from the test pile) is not over 1/4 inch, the allowable bearing value shall be one-half of the maximum load in the test. This rule is the most conservative that I have found, after examining the specifications of various highway departments and the building codes of some large cities.*** Fig. #5 is a graph of a typical pile load test in fine sand and shows a comparison of allowable bearing values from some of these specifications and codes. It should be remembered that a building code should be more conservative than a highway bridge specification in this matter, as a building is usually more delicate than a bridge. Also, it would seem that if spread footings are allowed to settle an inch or even several inches, a footing on piles should not be restricted to less than 1/4" settlement, unless the structure is very unusual.

Therefore, I believe the A.A.S.H.O. specification should be modernized by changing the 1/4" net settlement to read, "1/2" net settlement" like the recently approved Boston building code, or better, to something like the New York City code which states the allowable load is the smaller of the following: one-half the load when gross settlement is 1" or one-half the load causing a net settlement of 0.01" per ton of test load.**** These are shown in Fig. 5.

The other improvement in the A.A.S.H.O. pile specifications which seems desirable is in Article 1.4.17 - Case B (1) (d) which deals with end-bearing steel H-piles. Experience would indicate that a higher stress value than 6,000 psi. can be used without requiring a pile load test when the pile is driven to virtual refusal. In Connecticut we have been using

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* "Measurement of Forces Produced in Piles by Settlement of Adjacent Soil", by Gant, Stephens & Moulton, Highway Research Board Bulletin No. 173.

** "Standard Specifications for Highway Bridges", American Association of State Highway Officials, 1961.

***"Pile Foundations", by R. D. Chellis (second edition, 1961) McGraw-Hill Book Co., has a wealth of information on the subject of piles.

**** New York City Building Code, Sect. C26-405.2 (1948)

$10{,}000 \frac{\#}{in^2} \times \frac{144}{FT^2} = \frac{144\,K}{FT^2} = 72\,TSF$

design values of 9,000 psi. and even 12,000 psi. in recent years without apparent difficulty. Our load tests to twice design value in these cases indicate that gross settlement is nearly 100% due to elastic shortening of the pile, and net settlement is virtually zero. The large-scale load tests on H-piles made by the Armour Research Foundation for the U. S. Steel Corp. supply much information on this subject.*

Group #4 - Scour.

This subject does not normally involve soil mechanics, except that a knowledge of soil behavior, as well as hydraulics, is most helpful in studying scour. Past performance at the site and elsewhere, revealed by records and recollections of local residents are probably the best source of information, when reliable. However, in Connecticut, we have developed a Scour Formula** for small bridges, of 60 ft. span or less, based on measurements and studies of the pertinent data at many of our bridges during and immediately after the disastrous floods of 1955. Accurate and consistent data at 18 bridges were used, with a Chezy formula as the basis of the analysis. The formula is not claimed to be precise but will predict depths of scour with an error of only a foot or two.

Scour is probably the most difficult thing to predict in the field of bridge foundations. Probably because of its complexity, there has been little research on this subject until the last 4 or 5 years. However, it is an extremely important matter and can bring disaster to a bridge foundation and the structure it supports. There are scores of examples of bridges, nearly all without piles, which were destroyed by scour, chiefly in cohesionless soils.

It should be mentioned that if sheeting (anchored to the footing) is used to reduce scour damage at abutments, lateral earth pressures behind that sheeting should be included in the pressures on the abutment structure.

Group #5- Ground Water and River Water.

The items under this group are concerned chiefly with questions of rotting of timber piles, with need for gravel fill to stabilize a "mushy" footing excavation, and with cofferdams. I will elaborate on cofferdams to say that soil mechanics, using a carefully-made flow-net (See Fig. 4), can predict with a fair degree of accuracy whether the excavation and footing can be made "in the dry" or whether a tremie concrete seal is necessary. If the former can be done, usually a gravel fill about 2 ft. thick is used in the analysis and is recommended by the Soils Engineer. In general, if the bottom of the footing is 18 to 20 ft. or less, below river level, the work can be done "in the dry", but if this distance is more than 23 ft., a tremie seal should be used. This is assuming sheeting for the cofferdam extends about 10 ft. below bottom of footing and the soil is sand. When analyzing the adequacy of a cofferdam, submitted by a contractor for approval, whether or not it is to be unwatered, the Soils Engineer should furnish the Bridge Engineer with estimates of lateral earth and water pressures. Lateral earth pressures are difficult to calculate, as the lateral movements (or deformations) of the sheeting, above and below the bottom of excavation, largely determine the pressures. Lateral water pressure when the cofferdam is unwatered can be approximated from the flow net; the latter shows how the water seeping through the soil gradually loses head as it travels under the sheeting and up to the bottom of excavation.

LOSS OF HEAD DUE TO SEEPAGE OR FRICTION ON THE SOIL GRAINS.

Group #6 - Effect of Roadway Fill behind Abutments.

Too often in the past, the effect of the approach fill on the bridge abutment was overlooked. When a bridge is founded on a thick clay deposit, as described earlier in this paper, more than half the bridge settlement is due to the weight of the approach fills (see Fig. 1). When a bridge is on piles driven into or through a compressible stratum, the effect of the approach fill may be serious or even disastrous. We are familiar with the situation, all too prevalent in the past, of bridge abutments on piles, tilting forward a foot or more because of large differential lateral pressures created in soft foundation soils when the approach fill was placed. We also have cases of bridges on piles which settled seriously

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* "Evaluation of Steel Beam Piling" (Final Report), Armour Research Foundation of Illinois Institute of Technology, Chicago, Illinois. December, 1954.

** "Report on Investigation of Scour at Bridges Caused by Floods of 1955", by Moulton, Belcher and Butler. Highway Research Board Abstracts for September 1957, Vol. 27, No. 8

when the approach roadway fill was added. This is due to added load being imposed on the substructure by the fill and to added compressive stresses in the soft material, causing the latter to settle and drag down the piles, as previously described under "Piles". These cases are now avoided or minimized by adding an extra span at each end with stub or spill-through abutments, or by removing the compressible material, or by treating it with sand drainage wells, or by using an overload of fill with a waiting period, or by using struts, or by a combination of these. In these analyses, estimates of the soil behavior, including settlements and lateral pressures and deformations, are used by the Soils Engineer.

A spectacular example of settlements caused by the approach fill occurred some years ago at an Eastern city. The abutment piles were driven to a satisfactory formula value in a clayey sand overlying a thick layer of soft clay. Settlements were small until the approach fill was placed; the fill had caused a settlement of about 21" at the wingwalls and 2" at the front of the abutment when the situation was corrected by underpinning, at a cost of about $150,000.*

Another example of movements occurred many years ago in Connecticut at a grade separation structure near a stream. The structure is on H-piles driven to rock. Nevertheless, the wingwall nearest the stream moved about 2" laterally and 2" longitudinally. The wingwall of a large box culvert in that area also moved several inches. Consequently, when the box culvert was extended some years later, a small temporary overload was placed at the new wingwalls before building them; this reduced the movement of the wingwalls to 2" after they were built. It now appears that a heavier overload would have been desirable to further decrease the movement. An obvious remark can be made here that piles driven to refusal will not move vertically, but they will move laterally if the soil conditions so dictate.

When the soil adjacent to piles is resistive to movement, it will provide considerable lateral resistance to movement of the piles. Consequently, when analyzing the forces necessary to resist lateral earth pressure against a wall, some of that pressure will be resisted by the soil in front of the piles. A conservative working rule is to use 5 tons/pile for H-piles and 4 tons/pile for concrete and timber piles; this includes all piles in the footing. A series of tests on this subject was made by the U. S. Army Corps of Engineers.** Various footings with various combinations of plumb and batter piles were used; the piles were timber, driven 30 ft. in sand. The tests indicate the above rule appears conservative. Obviously, more tests are needed to cover different conditions.

The benefits to a Highway Department from the soils work connected with bridge foundations are of two types. One type is tangible: savings in cost by avoiding over-conservative designs and by avoiding conventional design in exceptionally poor soil areas. The other type of benefit from soils work is the less tangible one of savings in time and worry during construction, reduction of claims by contractors for delays and reduction of costly corrective changes. The Soils Engineer has taken his place in the Highway Department organization as a necessary and useful engineer whose special knowledge and experience is aiding the Design and Construction Engineers to secure safe and economical structures and roadways.

* * *

Attachments:

Figures #1 through #5

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* "Observed Settlements of Highway Structures Due to Consolidation of Alluvial Clay", by E. S. Barber. Symposium on Consolidation Testing of Soils, Special Tech. Publication No. 126, Amer. Soc. for Testing Materials (1951).

** "Lateral Load Tests on Groups of Battered and Vertical Piles", by L. B. Feagin. Symposium on Lateral Load Tests on Piles, Spec. Tech. Publication No. 154, Amer. Soc. for Testing Materials (1953).

Doc. No. 87

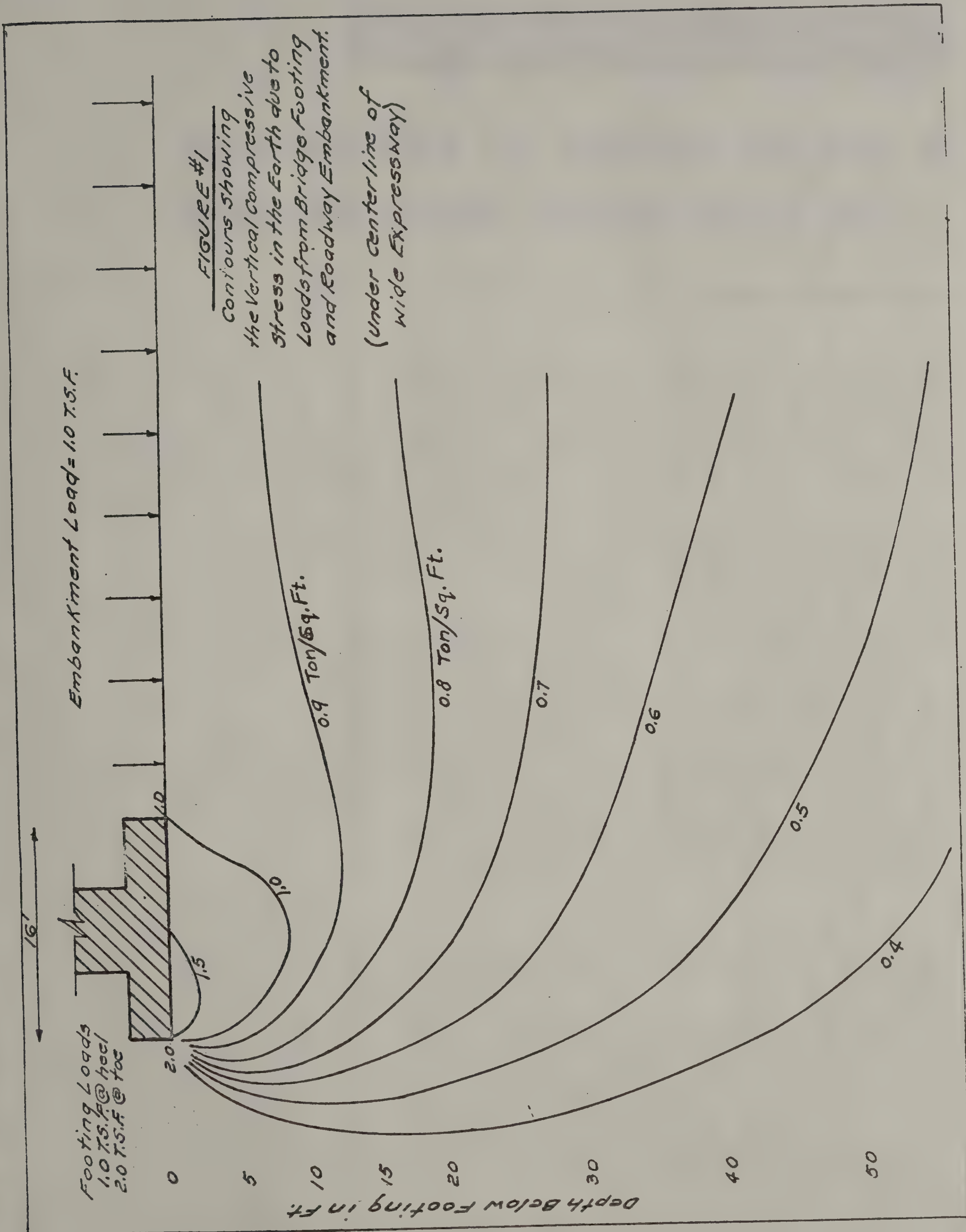

FIGURE #1

Contours Showing the Vertical Compressive Stress in the Earth due to Loads from Bridge Footing and Roadway Embankment.

(Under Center line of wide Expressway)

A graphic solution for soil pressure under eccentrically loaded footings

ALBERT W. KNOTT, A.M. ASCE, Assistant Professor of Civil Engineering, University of Colorado, Boulder, Colo.

A simple graphic solution is available for finding the theoretical distribution of soil pressure under an eccentrically loaded rectangular footing. This method is based on the graphic solution of the problem of locating the centroid of a trapezoid whose height and bases are known.

Given the trapezoid shown in Fig. 1 (*a*) of height *h*, and bases *a* and *b*, lay off the lengths *a* and *b* on their opposite bases, locating the points (1) and (2). Draw a line connecting these points.

Next connect the center of each of the bases, *a* and *b*, with a line (3)-(4), as shown in Fig. 1 (*b*). Point (5), the intersection of lines (1)-(2) and (3)-(4), is the centroid of the trapezoid. The proof of this construction is a simple problem in geometry.

In applying this solution to the problem of determining footing pressures, the engineer knows the size of the footing and the magnitude and location of the resultant of the applied loads. In determining the soil pressure distribution, the process is to reverse the above graphic solution. Given the average ordinate and the location of the centroid of the trapezoid, the lengths of its bases can be found.

Dividing the applied load by the area of the footing, the average ordinate, P/A, of the trapezoid is determined. Laying this dimension off as shown in Fig. 2, a point (1) is established. Through this point the pressure-distribution line, which by theory is a straight line, must pass. The problem that remains is to determine the slope of this line.

To find this slope two trial approximations of known centroid will be made, and the resulting error plotted. The plot will be used to determine the point of zero error, thus establishing the slope.

As a first trial the line (0)-(1)-(2), shown in Fig. 3 (*a*), is assumed to represent the pressure distribution. This produces a triangle whose centroid is known to be at the third point. This third point may be located in the same fashion as the centroid of the trapezoid in the original solution, or merely measured directly. If the triangle were the correct distribution, its centroid and the load centroid would have coincided. However, generally they do not coincide, and the error is noted in Fig. 3 (*a*) to be e_1, to the right of the load centroid. This error is laid off to the right from point (2), as in Fig. 3 (*b*), establishing a point (3).

In a similar fashion the second approximation is chosen to be (0′)-(1)-(4), as shown in Fig. 4 (*a*). This establishes a second error, e_2, to the left of the load line. Laying this error off to the left from point (0′) establishes a point (5), shown in Fig. 4 (*b*). Drawing the line (3)-(5), a point *E* is established where line (3)-(5) crosses line (0′)-(2). This point *E*, where the "error" is zero, determines the length of the right base of the correct pressure trapezoid. Points *E* and (1) establish the slope of the line, giving the final soil pressure distribution to scale. The result is shown in Fig. 5.

A rigorous proof of this solution is not presented here as the only variation from normal procedure was the drawing of the line (3)-(5) to locate *E*. This construction is familiar to students of graphic analysis. The validity of the construction is evident.

Two facts are of additional interest. If the load centroid is outside the kern, the graphic solution will still satisfactorily indicate the tensile stresses predicted by statics. Point *E*, however, will not fall between points (0′) and (2). Second, the triangles were used only because their centroids were easily located. It is just as convenient to use the P/A line in Fig. 2, with its centroid at the midpoint of the footing. Any approximation is valid.

My experience in using this graphic procedure in practice is that it is reasonably fast, and often more convenient than the normal algebraic approach. Also, the resulting stress diagram is in a more usable, understandable form.

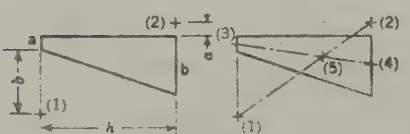

FIG. 1 (a) FIG. 1 (b)

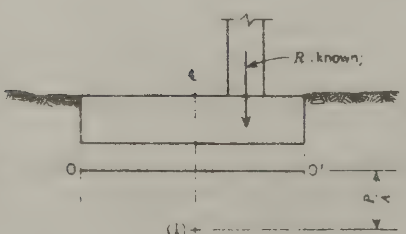

FIG. 2

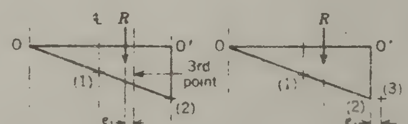

FIG. 3 (a) FIG. 3 (b)

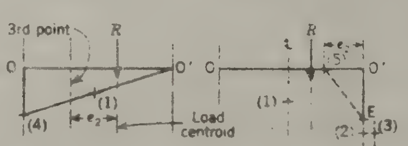

FIG. 4 (a) FIG. 4 (b)

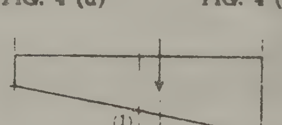

FIG. 5

11- 16

MALONE
DIST. 7
DIST. 2
DIST 1
UTICA
DIST. 9
ALBANY
DIST 8
VT
MASS
CONN

ENGINEERS' NOTEBOOK

A graphic solution for soil pressure under eccentrically loaded footings

ALBERT W. KNOTT, A.M. ASCE, Assistant Professor of Civil Engineering, University of Colorado, Boulder, Colo.

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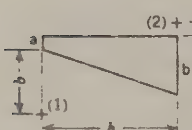


FIG. 1 (a)

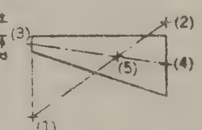


FIG. 1 (b)

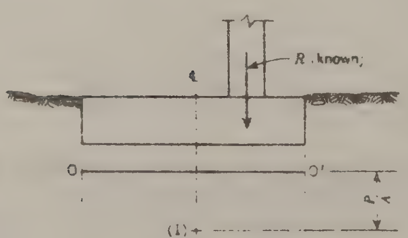


FIG. 2

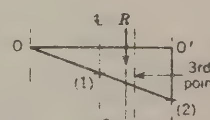


FIG. 3 (a)

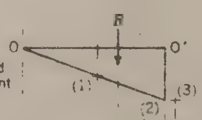


FIG. 3 (b)

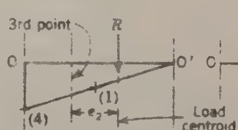


FIG. 4 (a)

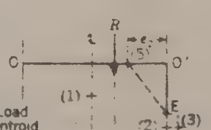
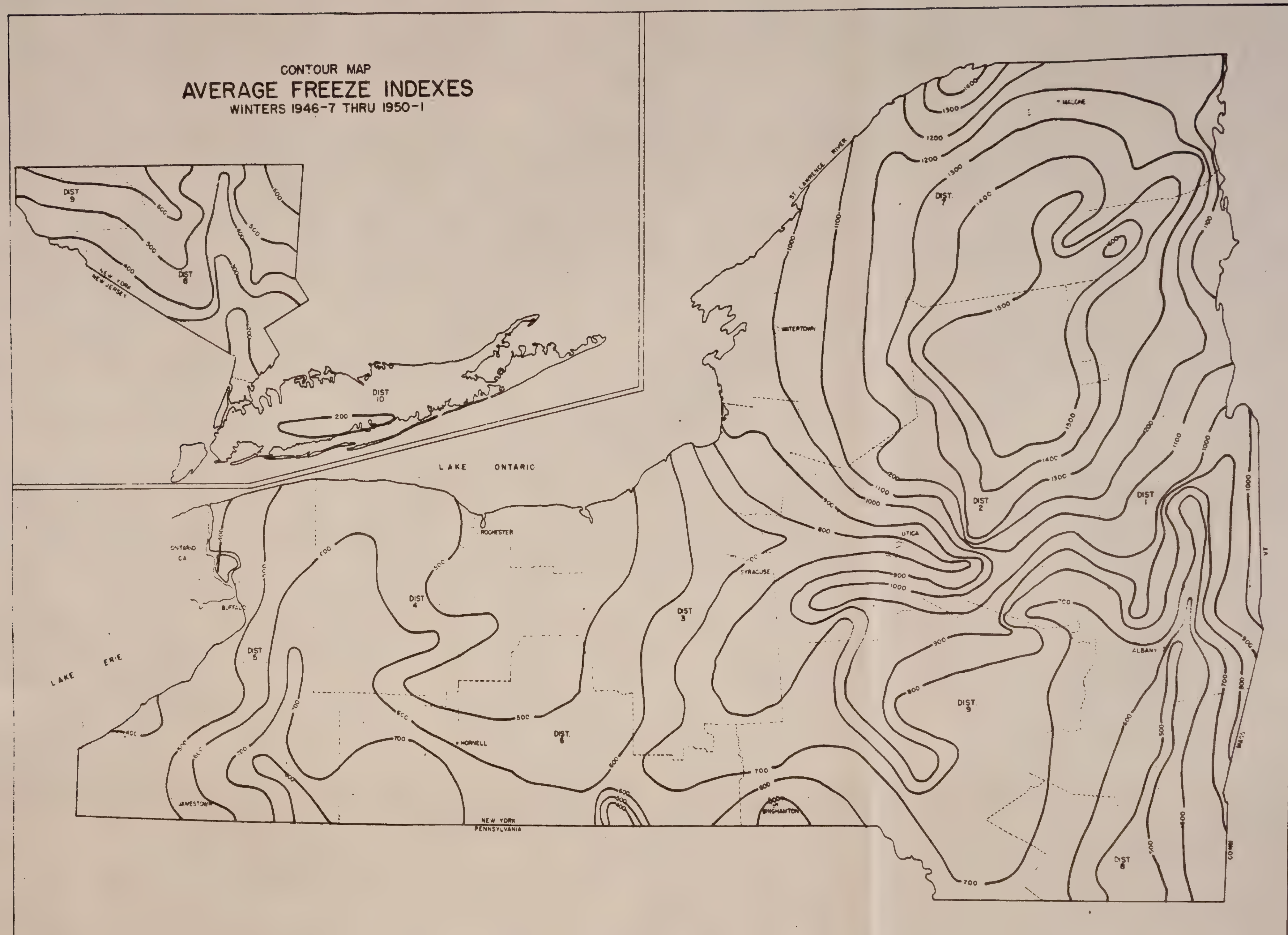


FIG. 4 (b)

FIG. 5


CONTOUR MAP
AVERAGE FREEZE INDEXES
WINTERS 1946-7 THRU 1950-1
LAKE ONTARIO
LAKE ERIE
ST. LAWRENCE RIVER
WATERTOWN
ROCHESTER
SYRACUSE
UTICA
ALBANY
HORNELL
JAMESTOWN
BINGHAMTON
BUFFALO
ONTARIO
CA
NEW YORK
PENNSYLVANIA
NEW YORK
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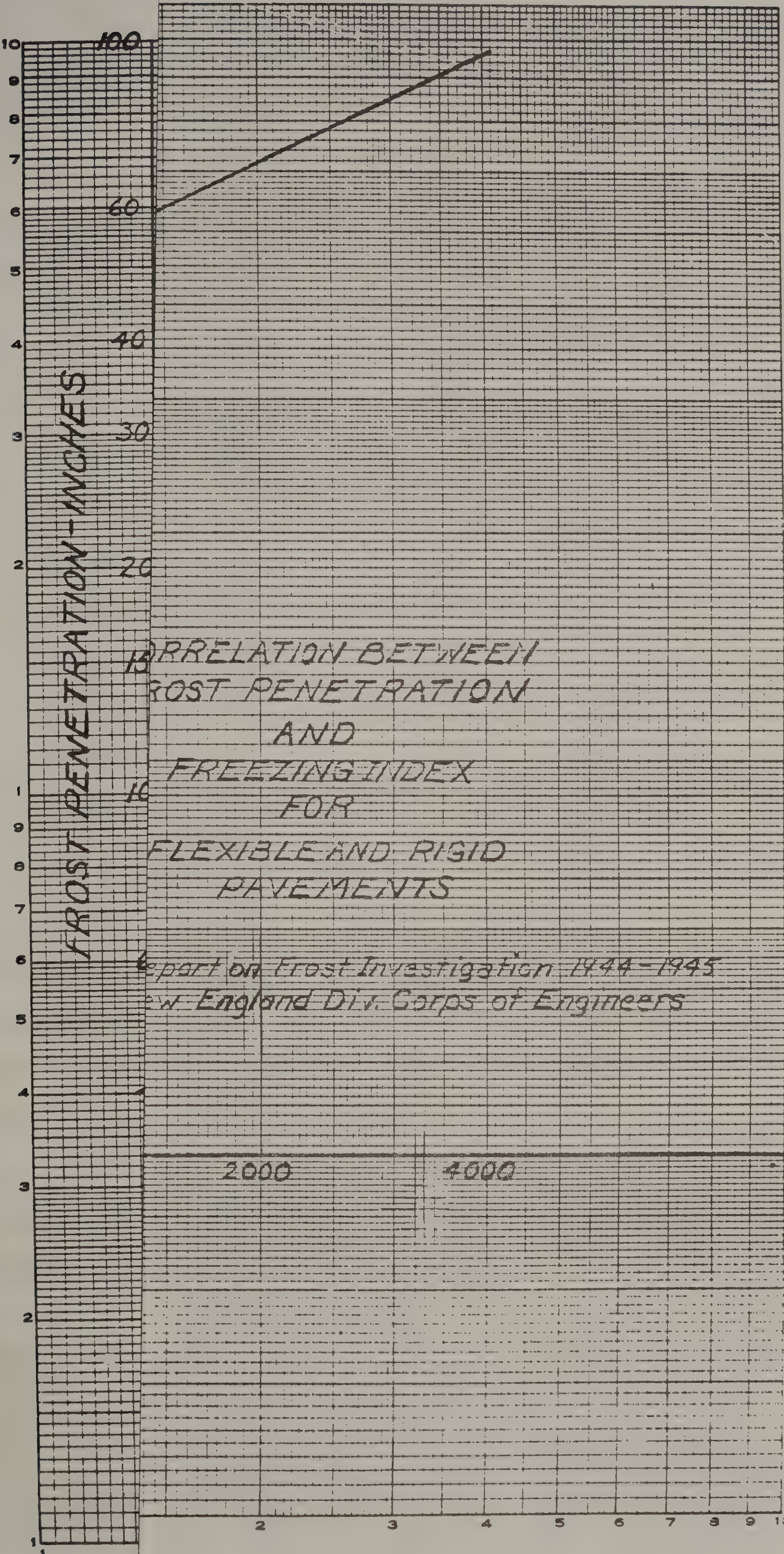
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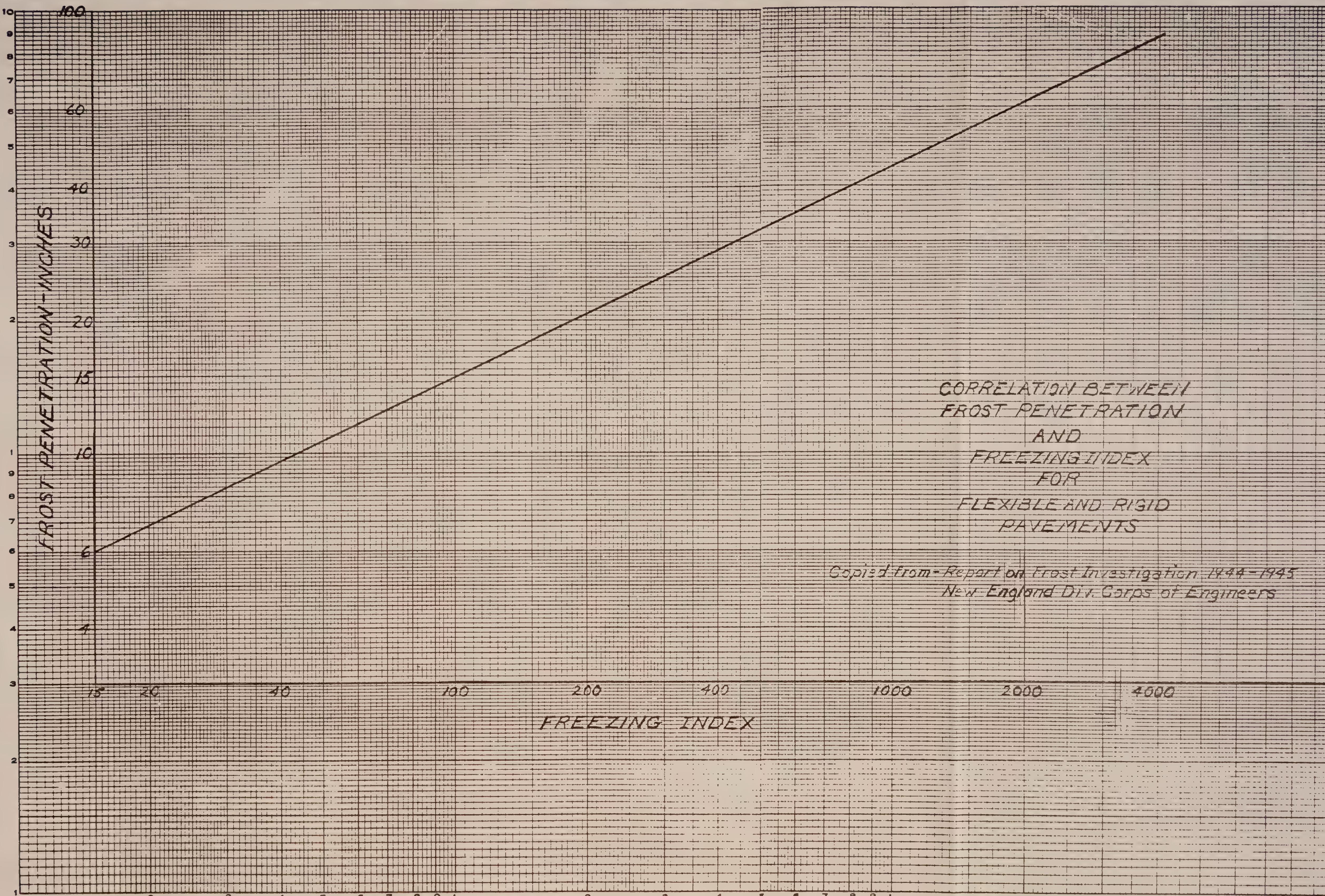
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FREEZING INDEX
CORRELATION BETWEEN
FROST PENETRATION
AND
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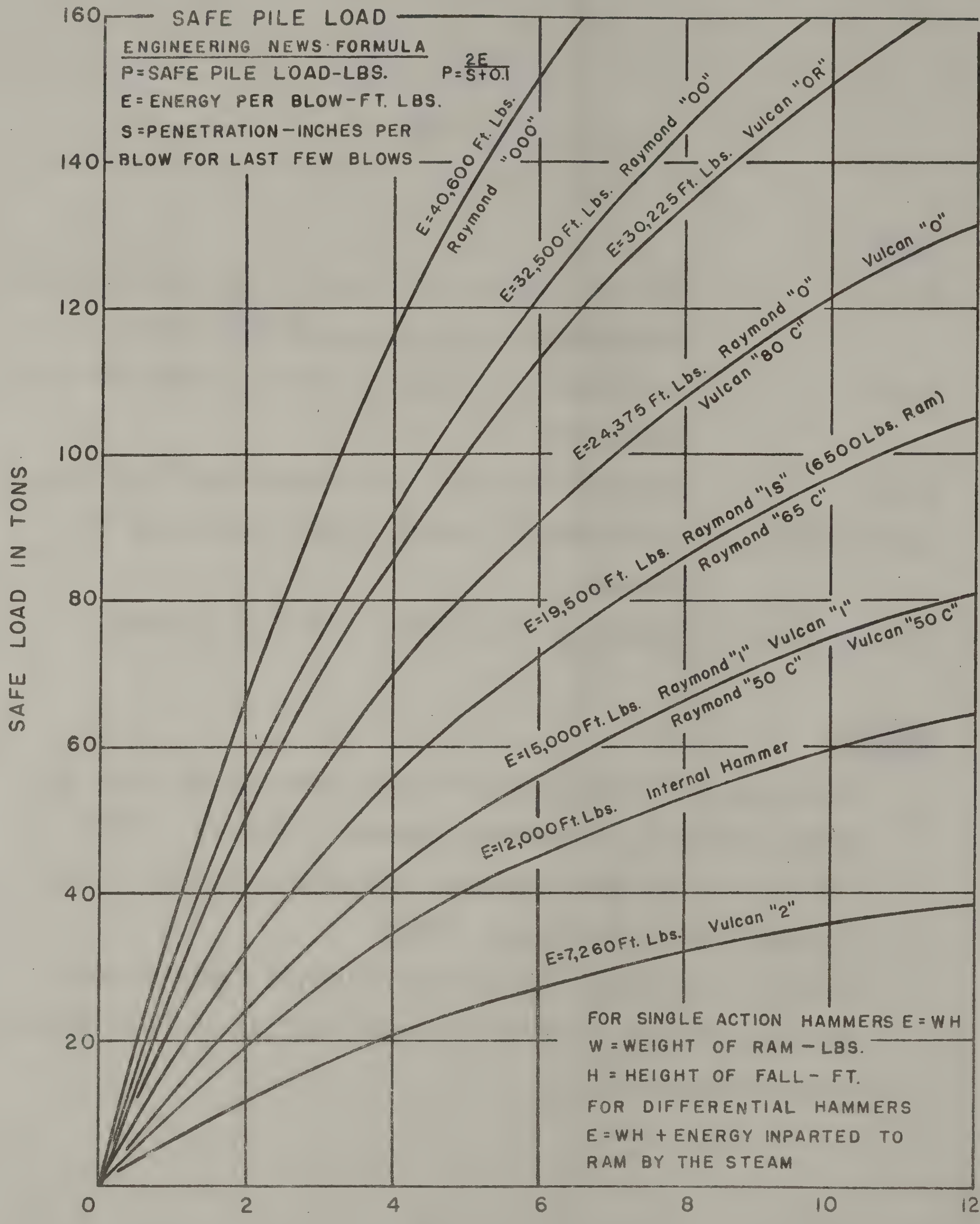
FOR
FLEXIBLE AND RIGID
PAVEMENTS
Copied from - Report on Frost Investigation 1944-1945
New England Div. Corps of Engineers

SECTION 13

PILE FOUNDATIONS



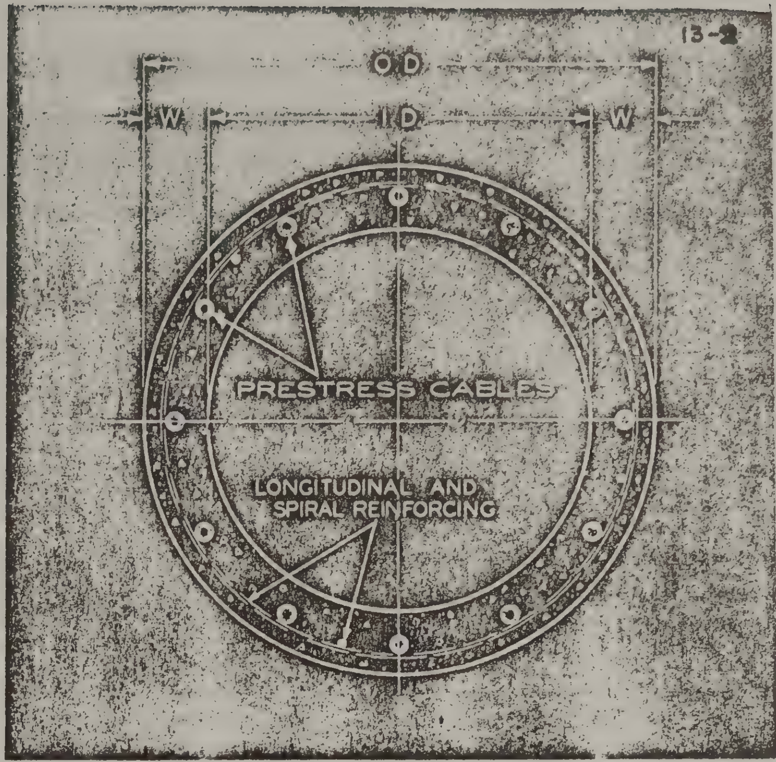
SINGLE ACTING AND

PRESTRESSED CYLINDRICAL PILES

STANDARD SIZES

O.D.	W	NUMBER OF PRESTRESSING CABLES
36″	4″ and 4½″	8 - 12 or 16
54″	4½″ and 5″	12 - 16 or 24

While our present output is standardized to the two sizes given in the above table, other sizes and wall thicknesses, as listed in the table below, can be furnished, provided the project is of sufficient size to warrant the purchase of the special equipment required.



★ STANDARD SIZES

PROPERTIES FOR DESIGN

SIZE			AREA	I	S	r	CIRCUMFERENCE	VOLUME PER FOOT	WEIGHT PER FOOT	CONCRETE DESIGN STRESS PER CABLE
O.D.	I.D.	WALL THICKNESS								
in	in	in	in²	in⁴	in³	in	in	Ft.³	Lbs.	LBS. PER SQ. INCH
24	16	4	251	13060	1089	7.2	75	1.74	262	166.0
	15	4½	276	13810	1150	7.1	75	1.92	288	150.5
	14	5	298	14380	1198	6.9	75	2.07	310	139.5
30	22	4	327	28200	1880	9.3	94	2.27	341	127.2
	21	4½	361	30170	2010	9.1	94	2.50	376	115.0
	20	5	393	31800	2120	9.0	94	2.73	410	106.0
36	28	4	402	52200	2900	11.4	113	2.79	419	103.5
	27	4½	445	56400	3130	11.3	113	3.09	463	93.6
	26	5	487	60000	3330	11.1	113	3.38	507	85.5
42	34	4	477	87000	4140	13.5	132	3.32	497	87.3
	33	4½	530	94800	4520	13.4	132	3.68	551	78.6
	32	5	581	102000	4850	13.2	132	4.04	605	71.7
	30	6	679	113300	5390	12.9	132	4.72	708	61.3
48	40	4	553	134900	5620	15.6	151	3.84	576	75.3
	39	4½	615	146700	6100	15.4	151	4.27	641	67.7
	38	5	675	158100	6600	15.3	151	4.69	703	61.7
	36	6	792	178000	7400	15.0	151	5.50	826	52.6
54	46	4	628	197400	7320	17.7	170	4.36	655	66.4
	45	4½	700	216000	8000	17.6	170	4.86	728	59.5
	44	5	770	233000	8630	17.4	170	5.35	802	54.2
	42	6	704	264000	9770	17.1	170	6.28	941	46.1

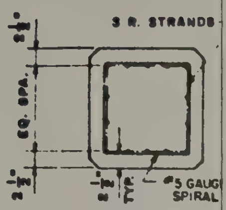
SMOOTH PIPE PILES

TABLE 1—DIMENSIONS* AND PROPERTIES OF ARMCO FOUNDATION PIPE

Size O.D. (ins.)	Wall Thickness (ins.)	Weight per lin. ft. (lbs.)	Moment of Inertia (ins.^{4})	Radius of Gyration (ins.)	Section Modulus (ins.^{3})	Area External Surface (sq. ft. per lin. ft.)	Area of Steel in Cross Section		Inside Cross Sectional Area (sq. in.)	Concrete per lin. ft. pipe (cu. yds.)	Approx. Collapsing Pressure (psi)
							Gross (sq. in.)	Wall less 1/16" (sq. in.)			
10	.141	14.81	52.90	3.49	10.58	2.62	4.367	2.415	74.17	.0191	190
	.164	17.27	61.34	3.48	12.27	2.62	5.080	3.128	73.46	.0189	300
	.172	18.04	64.10	3.48	12.82	2.62	5.311	3.359	73.23	.0189	340
	.179	18.81	66.60	3.47	13.32	2.62	5.532	3.580	73.01	.0188	380
	.188	19.70	69.60	3.47	13.92	2.62	5.795	3.843	72.74	.0187	440
	.219	22.88	80.45	3.45	16.09	2.62	6.730	4.778	71.81	.0185	680
	.250	26.03	91.05	3.45	18.21	2.62	7.658	5.706	70.88	.0182	1000
10¾	.141	15.93	65.95	3.75	12.27	2.81	4.699	2.600	86.06	.0221	150
	.164	18.59	76.50	3.74	14.23	2.81	5.470	3.371	85.30	.0219	240
	.172	19.42	79.93	3.74	14.87	2.81	5.716	3.617	85.05	.0219	270
	.179	20.24	83.09	3.74	15.44	2.81	5.960	3.861	84.80	.0218	310
	.188	21.15	86.81	3.74	16.15	2.81	6.238	4.139	84.52	.0217	350
	.203	22.88	93.39	3.73	17.38	2.81	6.726	4.627	84.04	.0216	440
	.219	24.60	100.41	3.72	18.68	2.81	7.245	5.146	83.52	.0215	550
	.250	28.04	113.74	3.71	21.16	2.81	8.247	6.148	82.52	.0212	820
	.279	31.20	126.85	3.70	23.60	2.81	9.242	7.143	81.52	.0210	1130
12	.141	17.81	92.16	4.19	15.36	3.14	5.253	2.908	107.84	.0277	110
	.164	20.78	106.85	4.18	17.81	3.14	6.113	3.768	106.98	.0275	180
	.172	21.71	111.72	4.18	18.62	3.14	6.391	4.046	106.71	.0274	200
	.179	22.60	116.12	4.18	19.35	3.14	6.659	4.314	106.44	.0274	220
	.188	23.72	121.38	4.18	20.23	3.14	6.976	4.631	106.12	.0273	260
	.203	25.57	130.78	4.17	21.80	3.14	7.529	5.184	105.57	.0272	320
	.219	27.56	140.52	4.17	23.42	3.14	8.105	5.760	105.00	.0270	400
	.250	31.37	159.36	4.16	26.56	3.14	9.228	6.883	103.87	.0267	590
	.281	35.17	177.90	4.14	29.65	3.14	10.345	8.000	102.75	.0264	830
	.312	38.95	195.78	4.13	32.63	3.14	11.456	9.111	101.64	.0261	1110
12¾	.141	18.94	110.73	4.46	17.37	3.34	5.585	3.093	122.09	.0314	95
	.164	22.10	128.49	4.45	20.15	3.34	6.500	4.009	121.18	.0312	150
	.172	23.09	134.39	4.45	21.08	3.34	6.797	4.305	120.88	.0311	170
	.179	24.07	139.64	4.44	21.91	3.34	7.081	4.589	120.60	.0310	190
	.188	25.16	146.05	4.44	22.91	3.34	7.419	4.927	120.26	.0309	220
	.203	27.20	157.34	4.44	24.68	3.34	8.008	5.516	119.67	.0308	270
	.219	29.28	169.13	4.43	26.53	3.34	8.621	6.129	119.06	.0306	340
	.250	33.38	191.82	4.42	30.09	3.34	9.818	7.326	117.86	.0303	500
	.281	37.45	214.26	4.41	33.61	3.34	11.008	8.516	116.67	.0300	700
	.312	41.51	236.26	4.40	37.06	3.34	12.191	9.699	115.49	.0297	950
14	.141	20.82	147.00	4.90	21.00	3.67	6.139	3.402	147.80	.0380	75
	.164	24.29	170.72	4.89	24.39	3.67	7.146	4.409	146.79	.0378	110
	.172	25.38	178.50	4.89	25.50	3.67	7.472	4.735	146.47	.0377	130
	.179	26.47	189.40	4.89	27.06	3.67	7.785	5.048	146.15	.0376	140
	.188	27.66	194.04	4.88	27.72	3.67	8.158	5.421	145.78	.0375	160
	.219	32.20	224.84	4.87	32.13	3.67	9.482	6.745	144.46	.0372	260
	.250	36.71	255.36	4.86	36.48	3.67	10.799	8.062	143.14	.0868	380
	.281	41.21	283.08	4.85	40.44	3.67	12.111	9.374	141.83	.0365	530
	.312	45.68	314.86	4.84	44.98	3.67	13.417	10.680	140.52	.0362	720
16	.172	29.06	267.68	5.596	33.46	4.19	8.553	5.423	192.51	.0495	90
	.179	30.30	278.37	5.594	34.80	4.19	8.912	5.782	192.15	.0494	100
	.188	31.66	291.12	5.591	36.39	4.19	9.339	6.210	191.72	.0493	110
	.219	36.87	337.76	5.580	42.22	4.19	10.858	7.728	190.20	.0489	170
	.250	42.05	383.68	5.569	47.96	4.19	12.370	9.240	188.69	.0485	260
	.281	47.22	429.12	5.558	53.64	4.19	13.877	10.748	187.19	.0482	360
	.312	52.36	473.92	5.548	59.24	4.19	15.377	12.247	185.69	.0477	490
	.375	62.58	562.08	5.526	70.26	4.19	18.408	15.278	182.65	.0470	830
18	.219	41.54	483.03	6.287	53.67	4.71	12.234	8.711	242.24	.0630	120
	.250	47.39	549.09	6.276	61.01	4.71	13.941	10.418	240.53	.0619	180
	.281	53.22	614.52	6.266	68.28	4.71	15.642	12.119	238.83	.0614	260
	.312	59.03	679.23	6.255	75.47	4.71	17.337	13.814	237.13	.0610	350
	.375	70.59	806.58	6.233	89.62	4.71	20.764	17.241	233.71	.0601	590
20	.250	52.73	756.50	6.983	75.65	5.24	15.512	11.596	298.65	.0768	130
	.281	59.23	847.10	6.972	84.71	5.24	17.408	13.492	296.75	.0763	190
	.312	65.71	936.70	6.962	93.67	5.24	19.298	15.382	294.86	.0758	260
	.375	78.60	1113.50	6.940	111.35	5.24	23.120	19.204	291.04	.0749	440
24	.250	63.41	1315.44	8.397	109.62	6.28	18.653	13.952	433.74	.1115	80
	.281	71.25	1474.20	8.386	122.85	6.28	20.939	16.238	431.45	.1110	110
	.312	79.06	1631.52	8.376	135.96	6.28	23.218	18.517	429.17	.1104	150
	.375	94.62	1942.44	8.354	161.87	6.28	27.833	23.132	424.56	.1092	260
	.500	125.49	2549.64	8.310	212.47	6.28	36.914	32.213	415.48	.1069	590
30	.312	99.08	3211.80	10.497	214.12	7.85	29.100	23.220	677.60	.1743	80
	.375	118.65	3829.95	10.475	255.33	7.85	34.901	29.021	672.00	.1728	130
	.500	157.53	5043.00	10.431	336.20	7.85	46.339	40.459	660.52	.1699	310
36	.312	119.11	5578.02	12.618	309.89	9.42	34.981	27.923	982.90	.2528	60
	.375	142.68	6658.74	12.596	369.93	9.42	41.970	34.912	975.91	.2510	80
	.500	189.57	8785.98	12.552	488.11	9.42	55.763	48.705	962.12	.2475	180

*Sizes listed here are standard. However, the Armco manufacturing process permits the making of other diameters from 6" to 36", and many other wall thicknesses.

S



Pile Size Diameter (1)	Area Ac
10"	100 sq. in.
12"	144 " "
14"	196 " "
16"	256 " "
18"	324 " "
20"	400 " "
22"	484 " "
24"	576 " "
20" HC	305 " "
22"HC	351 " "
24"HC	399 " "

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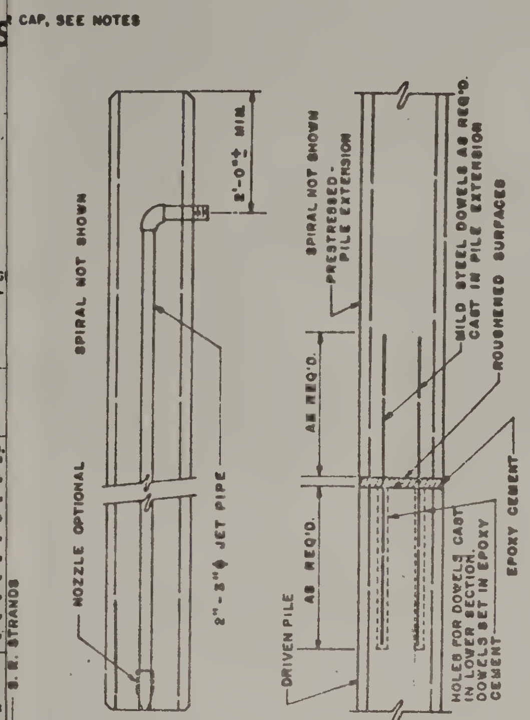


JET PIPE DETAILS

DOWELED SPLICE

NOTES:

of reinforcing steel shall be 1-1.2% of the gross cross-section of con- bars shall be in a symmetrical pattern of not less than four bars.

nt of pile to build-up may be by any of the methods given in the notes heads. If mild reinforcing steel is used for attachment, the area shall used in the build-up.

half of pile shall be bush-hammered to prevent feather edges.

or form may be rounded, flat or tapered with proper taping to prevent

PICKING POINTS

Pile Size Diameter (1)	Area Ac
10"	83 sq. in.
12"	119 " "
14"	162 " "
16"	212 " "
18"	268 " "
20"	331 " "
22"	401 " "
24"	477 " "
20"HC	236 " "
22"HC	268 " "
24"HC	300 " "

UP

0.21L

UP

NOTE

Unless special lifting devices are attached for pick-up, pick-up points shall be plainly marked on all piles after removal of the forms and all lifting shall be done at these points.

The use of special embedded or attached lifting devices, the employment of other pick-up points or any other method of pick-up shall be subject to approval by the Engineer.

(1)
(2)
(3)
(4)
(5)
(6)

JOINT COMMITTEE
ERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS
COMMITTEE ON BRIDGES & STRUCTURES
AND
PRESTRESSED CONCRETE INSTITUTE

D PRESTRESSED CONCRETE PILES
",12",14",16",18",20",22"&24"

Y

W. Jennings FOR AASHO

J. Lyman FOR PCI

SMOOTH PIPE PILES

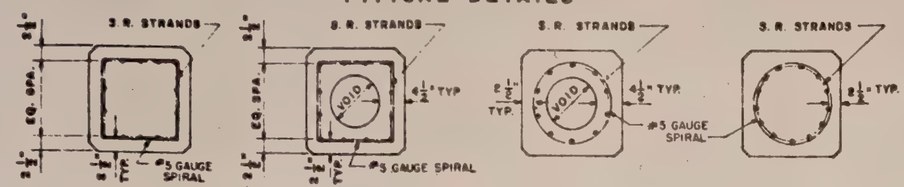
TABLE 1—DIMENSIONS* AND PROPERTIES OF ARMCO FOUNDATION PIPE

Size O.D. (ins.)	Wall Thickness (ins.)	Weight per lin. ft. (lbs.)	Moment of Inertia (ins.⁴)	Radius of Gyration (ins.)	Section Modulus (ins.³)	Area External Surface (sq. ft. per lin. ft.)	Area of Steel in Cross Section: Gross (sq. in.)	Area of Steel in Cross Section: Wall less 1/16" (sq. in.)	Inside Cross Sectional Area (sq. in.)	Concrete per lin. ft. pipe (cu. yds.)	Approx. Collapsing Pressure (psi)
10	.141	14.81	52.90	3.49	10.58	2.62	4.367	2.415	74.17	.0191	190
	.164	17.27	61.34	3.48	12.27	2.62	5.080	3.128	73.46	.0189	300
	.172	18.04	64.10	3.48	12.82	2.62	5.311	3.359	73.23	.0189	340
	.179	18.81	66.60	3.47	13.32	2.62	5.532	3.580	73.01	.0188	380
	.188	19.70	69.60	3.47	13.92	2.62	5.795	3.843	72.74	.0187	440
	.219	22.88	80.45	3.45	16.09	2.62	6.730	4.778	71.81	.0185	680
	.250	26.03	91.05	3.45	18.21	2.62	7.658	5.706	70.88	.0182	1000
10¾	.141	15.93	65.95	3.75	12.27	2.81	4.699	2.600	86.06	.0221	150
	.164	18.59	76.50	3.74	14.23	2.81	5.470	3.371	85.30	.0219	240
	.172	19.42	79.93	3.74	14.87	2.81	5.716	3.617	85.05	.0219	270
	.179	20.24	83.09	3.74	15.44	2.81	5.960	3.861	84.80	.0218	310
	.188	21.15	86.81	3.74	16.15	2.81	6.238	4.139	84.52	.0217	350
	.203	22.88	93.39	3.73	17.38	2.81	6.726	4.627	84.04	.0216	440
	.219	24.60	100.41	3.72	18.68	2.81	7.245	5.146	83.52	.0215	550
	.250	28.04	113.74	3.71	21.16	2.81	8.247	6.148	82.52	.0212	820
	.279	31.20	126.85	3.70	23.60	2.81	9.242	7.143	81.52	.0210	1130
12	.141	17.81	92.16	4.19	15.36	3.14	5.253	2.908	107.84	.0277	110
	.164	20.78	106.85	4.18	17.81	3.14	6.113	3.768	106.98	.0275	180
	.172	21.71	111.72	4.18	18.62	3.14	6.391	4.046	106.71	.0274	200
	.179	22.60	116.12	4.18	19.35	3.14	6.659	4.314	106.44	.0274	220
	.188	23.72	121.38	4.18	20.23	3.14	6.976	4.631	106.12	.0273	260
	.203	25.57	130.78	4.17	21.80	3.14	7.529	5.184	105.57	.0272	320
	.219	27.56	140.52	4.17	23.42	3.14	8.105	5.760	105.00	.0270	400
	.250	31.37	159.36	4.16	26.56	3.14	9.228	6.883	103.87	.0267	590
	.281	35.17	177.90	4.14	29.65	3.14	10.345	8.000	102.75	.0264	830
	.312	38.95	195.78	4.13	32.63	3.14	11.456	9.111	101.64	.0261	1110
12¾	.141	18.94	110.73	4.46	17.37	3.34	5.585	3.093	122.09	.0314	95
	.164	22.10	128.49	4.45	20.15	3.34	6.500	4.009	121.18	.0312	150
	.172	23.09	134.39	4.45	21.08	3.34	6.797	4.305	120.88	.0311	170
	.179	24.07	139.64	4.44	21.91	3.34	7.081	4.589	120.60	.0310	190
	.188	25.16	146.05	4.44	22.91	3.34	7.419	4.927	120.26	.0309	220
	.203	27.20	157.34	4.44	24.68	3.34	8.008	5.516	119.67	.0308	270
	.219	29.28	169.13	4.43	26.53	3.34	8.621	6.129	119.06	.0306	340
	.250	33.38	191.82	4.42	30.09	3.34	9.818	7.326	117.86	.0303	500
	.281	37.45	214.26	4.41	33.61	3.34	11.008	8.516	116.67	.0300	700
	.312	41.51	236.26	4.40	37.06	3.34	12.191	9.699	115.49	.0297	950
14	.141	20.82	147.00	4.90	21.00	3.67	6.139	3.402	147.80	.0380	75
	.164	24.29	170.72	4.89	24.39	3.67	7.146	4.409	146.79	.0378	110
	.172	25.38	178.50	4.89	25.50	3.67	7.472	4.735	146.47	.0377	130
	.179	26.47	189.40	4.89	27.06	3.67	7.785	5.048	146.15	.0376	140
	.188	27.66	194.04	4.88	27.72	3.67	8.158	5.421	145.78	.0375	160
	.219	32.20	224.84	4.87	32.13	3.67	9.482	6.745	144.46	.0372	260
	.250	36.71	255.36	4.86	36.48	3.67	10.799	8.062	143.14	.0868	380
	.281	41.21	283.08	4.85	40.44	3.67	12.111	9.374	141.83	.0365	530
	.312	45.68	314.86	4.84	44.98	3.67	13.417	10.680	140.52	.0362	720
16	.172	29.06	267.68	5.596	33.46	4.19	8.553	5.423	192.51	.0495	90
	.179	30.30	278.37	5.594	34.80	4.19	8.912	5.782	192.15	.0494	100
	.188	31.66	291.12	5.591	36.39	4.19	9.339	6.210	191.72	.0493	110
	.219	36.87	337.76	5.580	42.22	4.19	10.858	7.728	190.20	.0489	170
	.250	42.05	383.68	5.569	47.96	4.19	12.370	9.240	188.69	.0485	260
	.281	47.22	429.12	5.558	53.64	4.19	13.877	10.748	187.19	.0482	360
	.312	52.36	473.92	5.548	59.24	4.19	15.377	12.247	185.69	.0477	490
	.375	62.58	562.08	5.526	70.26	4.19	18.408	15.278	182.65	.0470	830
18	.219	41.54	483.03	6.287	53.67	4.71	12.234	8.711	242.24	.0630	120
	.250	47.39	549.09	6.276	61.01	4.71	13.941	10.418	240.53	.0619	180
	.281	53.22	614.52	6.266	68.28	4.71	15.642	12.119	238.83	.0614	260
	.312	59.03	679.23	6.255	75.47	4.71	17.337	13.814	237.13	.0610	350
	.375	70.59	806.58	6.233	89.62	4.71	20.764	17.241	233.71	.0601	590
20	.250	52.73	756.50	6.983	75.65	5.24	15.512	11.596	298.65	.0768	130
	.281	59.23	847.10	6.972	84.71	5.24	17.408	13.492	296.75	.0763	190
	.312	65.71	936.70	6.962	93.67	5.24	19.298	15.382	294.86	.0758	260
	.375	78.60	1113.50	6.940	111.35	5.24	23.120	19.204	291.04	.0749	440
24	.250	63.41	1315.44	8.397	109.62	6.28	18.653	13.952	433.74	.1115	80
	.281	71.25	1474.20	8.386	122.85	6.28	20.939	16.238	431.45	.1110	110
	.312	79.06	1631.52	8.376	135.96	6.28	23.218	18.517	429.17	.1104	150
	.375	94.62	1942.44	8.354	161.87	6.28	27.833	23.132	424.56	.1092	260
	.500	125.49	2549.64	8.310	212.47	6.28	36.914	32.213	415.48	.1069	590
30	.312	99.08	3211.80	10.497	214.12	7.85	29.100	23.220	677.60	.1743	80
	.375	118.65	3829.95	10.475	255.33	7.85	34.901	29.021	672.00	.1728	130
	.500	157.53	5043.00	10.431	336.20	7.85	46.339	40.459	660.52	.1699	310
36	.312	119.11	5578.02	12.618	309.89	9.42	34.981	27.923	982.90	.2528	60
	.375	142.68	6658.74	12.596	369.93	9.42	41.970	34.912	975.91	.2510	80
	.500	189.57	8785.98	12.552	488.11	9.42	55.763	48.705	962.12	.2475	180

*Sizes listed here are standard. However, the Armco manufacturing process permits the making of other diameters from 6" to 36", and many other wall thicknesses.

SQUARE PRESTRESSED PILES

TYPICAL DETAILS

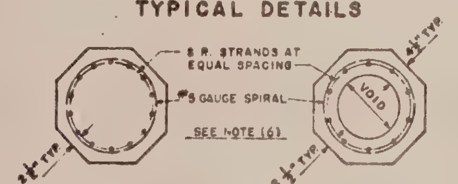


PILE PROPERTIES

Pile Size Diameter (1)	Area Ac	Approx. Weight per l/f (2)	Minimum Prestress Force (3)	Strands Per Pile Diameter 7/16" (4)	1/2"	Section Modulus	Perimeter	Design Bearing Capacity Concrete Strength 5000 psi (5)	6000 psi
10"	100 sq. in.	105#	70 kips	4	4	167 In.³	40 In.	50 tons	60 tons
12"	144 " "	150#	101 "	6	5	288 In.³	48 In.	72 "	86 "
14"	196 " "	205#	138 "	8	6	457 In.³	56 In.	98 "	117 "
16"	256 " "	265#	180 "	11	8	683 In.³	64 In.	128 "	153 "
18"	324 " "	335#	227 "	13	10	972 In.³	72 In.	162 "	194 "
20"	400 " "	415#	280 "	16	12	1333 In.³	80 In.	200 "	240 "
22"	484 " "	505#	339 "	20	15	1775 In.³	88 In.	242 "	290 "
24"	576 " "	600#	404 "	23	18	2304 In.³	96 In.	288 "	345 "
20" HC	305 " "	320#	214 "	13	10	1261 In.³	80 In.	152 "	183 "
22" HC	351 " "	365#	246 "	14	11	1647 In.³	88 In.	175 "	210 "
24" HC	399 " "	415#	280 "	16	12	2097 In.³	96 In.	200 "	240 "

OCTAGONAL PRESTRESSED PILES

TYPICAL DETAILS



PILE PROPERTIES

Pile Size Diameter (1)	Area Ac	Approx. Weight per l/f (2)	Minimum Prestress Force (3)	Strands Per Pile Diameter 7/16" (4)	1/2"	Section Modulus	Perimeter	Design Bearing Capacity Concrete Strength 5000 psi (5)	6000 psi
10"	83 sq. in.	85#	59 kips	4	4	111 in.³	34 in.	41 tons	50 tons
12"	119 " "	125#	84 "	5	4	189 "	40 "	59 "	71 "
14"	162 " "	170#	114 "	7	5	301 "	46 "	81 "	97 "
16"	212 " "	220#	149 "	9	7	449 "	54 "	106 "	127 "
18"	268 " "	280#	188 "	11	8	639 "	60 "	134 "	160 "
20"	331 " "	345#	232 "	14	10	877 "	66 "	165 "	198 "
22"	401 " "	420#	281 "	16	12	1167 "	72 "	200 "	240 "
24"	477 " "	495#	334 "	19	15	1515 "	80 "	238 "	286 "
20" HC	236 " "	245#	166 "	10	8	805 "	66 "	118 "	141 "
22" HC	268 " "	280#	188 "	11	8	1040 "	72 "	134 "	160 "
24" HC	300 " "	315#	210 "	12	9	1308 "	80 "	150 "	180 "

NOTES:

(For both square and octagonal piles)

(1) Voids in 20", 22" and 24" diameter hollow-core (HC) piles are 11", 13" and 15" diameter, respectively, providing a minimum 4-1/2" wall thickness. If a greater wall thickness is desired, properties should be increased accordingly.

(2) Weights based on 150 lb. per cubic foot of regular concrete.

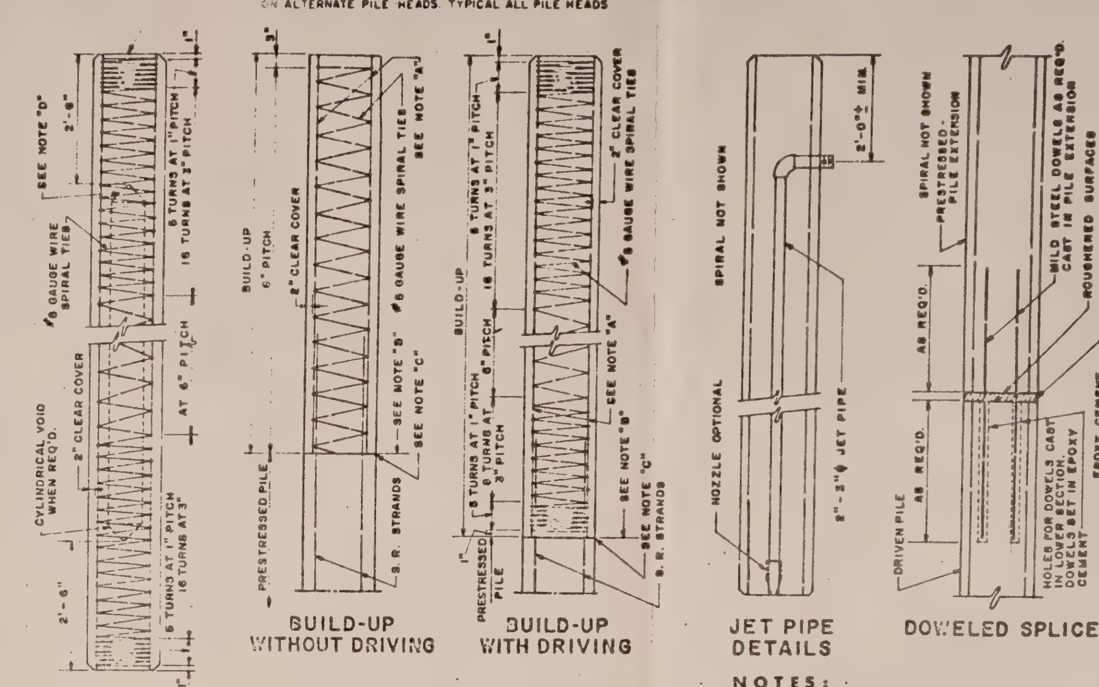
(3) Minimum prestress force based on unit prestress of 700 psi after losses.

(4) Based on 7/16" and 1/2" high strength strand with an ultimate strength of 31,000 lbs. and 41,300 lbs. respectively. If regular strength strand is used, the number of strands per pile should be increased accordingly in conformance with strand strand manufacturer's tables.

(5) Design bearing capacity based on 5000 psi and 6000 psi concrete and an allowable unit stress on the tip of the pile of .2 f'cAc. These bearing capacity values may be increased if higher strength concrete is used.

(6) Circular piles of the same diameter may be used in lieu of octagonal subject to Engineer's approval.

NOTE: FOR METHOD OF ATTACHMENT OF PILE HEAD TO FOOTING OR CAP, SEE NOTES ON ALTERNATE PILE HEADS. TYPICAL ALL PILE HEADS



NOTES:

Note A: The minimum area of reinforcing steel shall be 1-1/2% of the gross cross-section of concrete. Placement of bars shall be in a symmetrical pattern of not less than four bars.

Note B: Method of attachment of pile to build-up may be by any of the methods given in the notes on Alternate Pile Heads. If mild reinforcing steel is used for attachment, the area shall be no less than that used in the build-up.

Note C: Concrete around top half of pile shall be bush-hammered to prevent feather edges.

Note D: Conical end fitting or form may be rounded, flat or tapered with proper taping to prevent leakage.

ALTERNATE PILE HEADS

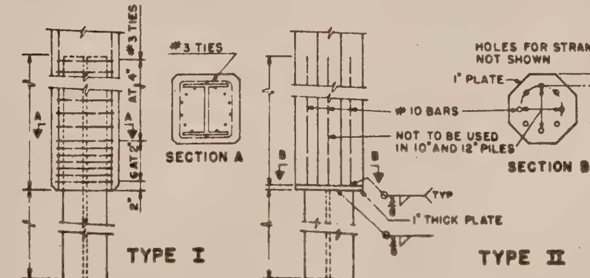
Reinforcement may be specified to project from the pile into the cap or footing. If so required, attachment of the pile to the cap or footing may be made by any one of the following methods unless otherwise specified:

1. Allow all strands to project a minimum of 24".
2. Cast mild reinforcing steel in pile head with bars projecting for anchorage.
3. Provide cored holes in pile head for subsequent use of grouted dowel bars.
4. Drill holes in pile head for installation of grouted dowel bars. Special care shall be taken to prevent damage to the pile head.

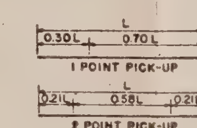
If mild reinforcing steel is used for projection into cap or footing, the minimum area of steel required shall be twice the area of the prestressing strands with not less than four bars being used. Arrangement of bars shall be in a symmetrical pattern with bars as close as practical to the sides of the pile. Anchorage of bars shall be sufficient to develop strength of bar but not less than 20 bar diameters.

ALTERNATE PILE TIPS

When driving into rock or hard strata, either Type I or Type II alternate tips may be used in lieu of the standard flat tip. Size and length of steel section used shall be as determined by Engineer for adequate penetration. Type I or Type II tips may be used for either square or octagonal piles.



PICKING POINTS



NOTE

Unless special lifting devices are attached for pick-up, pick-up points shall be plainly marked on all piles after removal of the forms and all lifting shall be done at these points.

The use of special embedded or attached lifting devices, the employment of other pick-up points or any other method of pick-up shall be subject to approval by the Engineer.

SHEET 1 OF 3

JOINT COMMITTEE
AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS
COMMITTEE ON BRIDGES & STRUCTURES
AND
PRESTRESSED CONCRETE INSTITUTE

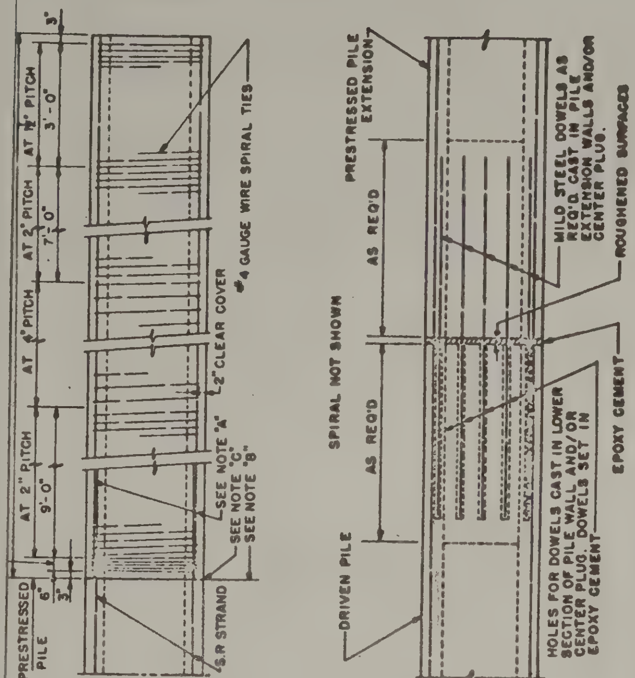
STANDARD PRESTRESSED CONCRETE PILES
10",12",14",16",18",20",22"&24"

SUBMITTED BY

T. W. Jennings FOR AASHO

R. J. Lyman FOR PCI

ƐAD TO FOOTING OR CAP,
YPICAL ALL PILE HEADS.



BUILD-UP
WITH DRIVING

DOWELED SPLICE

NOTES

area of reinforcing steel shall be 1-1/2% of the gross of concrete. Placement of bars shall be in a symmetrical less than eight bars.

achment of pile to build-up may be by any of the methods otes on Alternate Pile Heads. If mild reinforcing steel is chment, the area shall be no less than that used in the

nd top half of pile shall be bush-hammered to prevent

JOINT COMMITTEE
AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS
COMMITTEE ON BRIDGES & STRUCTURES
AND
PRESTRESSED CONCRETE INSTITUTE

PRESTRESSED CONCRETE CYLINDER PILES
36"-48"-54"

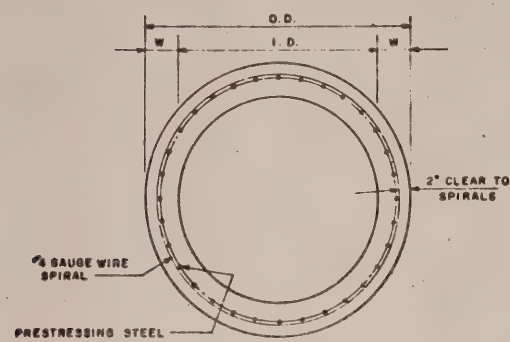
SUBMITTED BY

T. W. Jennings FOR AASHO

R. J. Lyman FOR PCI

PRETENSIONED CYLINDER PILES

TYPICAL DETAILS



PILE PROPERTIES

Pile Size OD	ID	W	Area Ac	Approx. Weight Per /f (1)	Minimum Prestress Force (2)	Strands Per Pile Diameter 7/16" (3)	1/2"	I	Section Modulus	Perimeter	Design Bearing Capacity Concrete Strength 5000 psi (4)	6000 psi
36 in.	26 in.	5 in.	487 sq. in.	508#	414 kips	24	18	60,000 in.⁴	3334 in.³	113 in.	242 tons	292 tons
	24 "	6 "	565 " "	590#	481 "	28	21	66,100 "	3676 "	113 "	282 "	339 "
48 in.	38 "	5 "	675 " "	703#	574 "	33	25	158,200 "	6593 "	151 "	337 "	405 "
	36 "	6 "	792 " "	826#	674 "	39	29	178,100 "	7422 "	151 "	396 "	475 "
54 in.	44 "	5 "	770 " "	802#	655 "	38	28	233,400 "	8645 "	170 "	385 "	462 "
	42 "	6 "	904 " "	940#	769 "	44	33	264,600 "	9802 "	170 "	452 "	542 "

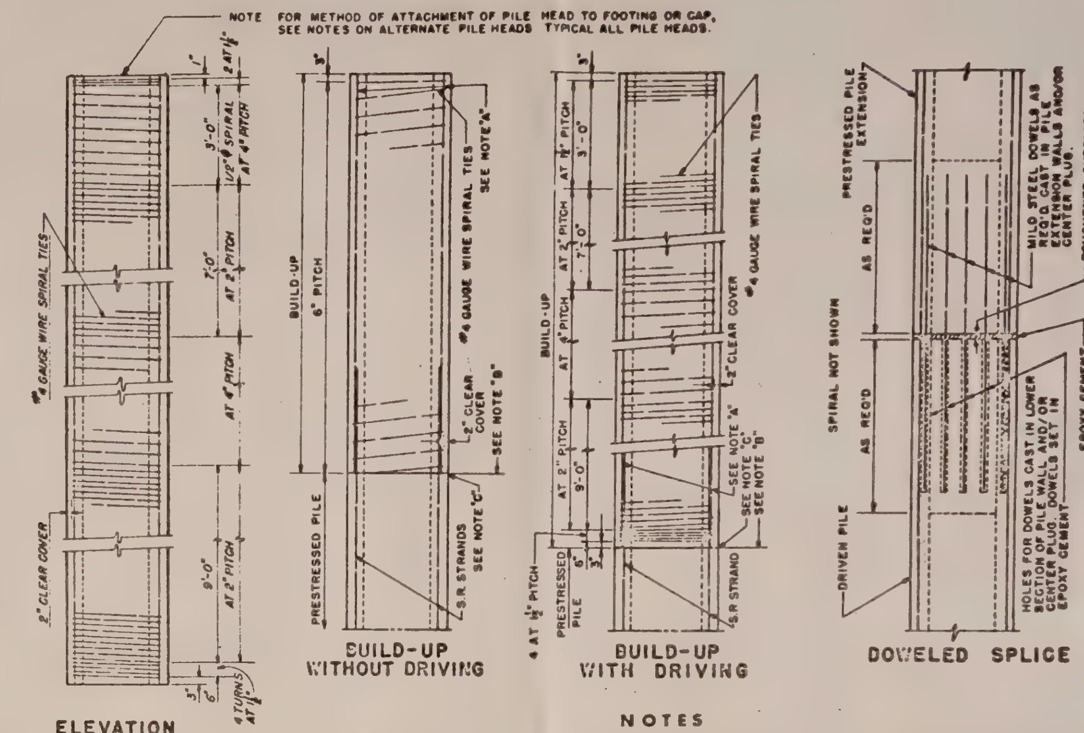
NOTES

(1). Weights are based on 150 lbs. per cubic foot of regular concrete.

(2). Minimum prestress force based on unit prestress of 850 psi after losses.

(3). Based on 7/16" and 1/2" high strength strand with an ultimate strength of 31,000 lbs. and 41,300 lbs. respectively. If regular strength strand is used, the number of strands per pile should be increased accordingly in conformance with strand manufacturer's tables.

(4). Design bearing capacity based on 5000 psi and 6000 psi concrete and an allowable unit stress on the tip of the pile of .2 f'cAc. These bearing capacity values may be increased if higher strength concrete is used.



NOTES

Note A: The minimum area of reinforcing steel shall be 1-1/2% of the gross cross-section of concrete. Placement of bars shall be in a symmetrical pattern of not less than eight bars.

Note B: Method of attachment of pile to build-up may be by any of the methods given in the notes on Alternate Pile Heads. If mild reinforcing steel is used for attachment, the area shall be no less than that used in the build-up.

Note C: Concrete around top half of pile shall be bush-hammered to prevent feather edges.

ALTERNATE PILE HEADS

Reinforcement may be specified to project from the pile into the cap or footing. If so required, attachment of the pile to the cap or footing may be made by any one of the following methods, unless otherwise specified:

1. Allow all strands to project a minimum of 24".
2. Cast mild reinforcing steel in plug poured in top of pile head, after driving, with bars projecting for anchorage.
3. If pile is driven with a plug in pile head, cored holes may be provided in plug for subsequent use of grouted dowel bars.

Plug in pile head may be cast in place or precast. If precast, plug shall be adequately bonded to inner wall of pile by an epoxy, a non-shrink grout or other means acceptable to the Engineer. Depth of plug in pile head shall not be less than the outside diameter of the pile.

If mild reinforcing steel is used for projection into the cap or footing, the minimum area of steel required shall be 1-1/2% of the gross cross section of concrete with not less than eight bars being used. Arrangement of bars shall be in a symmetrical pattern with bars as close as practical to the sides of the pile. Anchorage of bars shall be sufficient to develop strength of bar, but not less than 20 bar diameters.

JOINT COMMITTEE
AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS
COMMITTEE ON BRIDGES & STRUCTURES
AND
PRESTRESSED CONCRETE INSTITUTE

PRESTRESSED CONCRETE CYLINDER PILES
36"-48"-54"

SUBMITTED BY

T. W. Jennings FOR AASHO

R. J. Lyman FOR PCI

PILE CAPACITIES AND LOADING

(a) Loads on Piles as Columns – The maximum compressive stress on prestressed concrete piles in excess of the effective prestress shall not exceed the following:

fpc = 1000 for L/D = 0 to 12
fpc = 1240 – 20 L/D for L/D = 12 to 25
where L = effective length of pile
D = diameter or width of pile
fpc = allowable unit compressive stress in psi exclusive of effective prestress.

For piles considered hinged at both ends, "L" shall be taken as the actual length of pile. For piles considered hinged at one end and fully fixed at the other end, "L" shall be taken as 0.7 of the length between hinge and assumed location of fixity. For piles considered fully fixed at both ends, "L" shall be considered as 0.5 of the length between the two assumed fixed ends. When pile is designed as a cantilever, "L" shall be considered as 2 times the length of the cantilever.

Where it is necessary to use piles with an L/D greater than 25, they shall be thoroughly investigated for elastic stability using recognized formulae and applying a minimum factor of safety of two.

The above factor shall apply either for direct axial load or combination of axial load and bending. Where bending stresses occur, the maximum allowable tensile stress in concrete shall not exceed 250 psi.

The above factors are based upon a design concrete strength (f'c) of 5000 psi. For any higher values, these stresses may be increased in direct proportion to design concrete strengths.

If an effective prestress in excess of 0.2 f'c is used, the stresses as permitted above shall be reduced accordingly.

(b) Capacity in End Bearing of Prestressed Concrete Piles – The capacity of prestressed concrete piles in end bearing shall not exceed 0.2 f'c.

(c) Capacity of Prestressed Concrete Piles as Friction Piles – The capacity of prestressed concrete piles shall preferably be determined by loading tests. The capacity shall be limited to the lesser of either (a) above or the safe capacity resulting from test loading.

JOINT COMMITTEE
AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS
COMMITTEE ON BRIDGES & STRUCTURES
AND
PRESTRESSED CONCRETE INSTITUTE

GENERAL NOTES
PRESTRESSED CONCRETE PILES

UBMITTED BY

T. W. Jennings FOR AASHO

R. J. Lyman FOR PCI

GENERAL NOTES

PURPOSE: These standards have been revised to show the latest developments in materials and uses of prestressed concrete piles. In addition to square and octagonal piles, details for pretensioned cylinder piles have been added to these standards.

Any other shape of pretensioned concrete piles, or post-tensioned cylinder piles, with similar properties, may be used subject to approval of the Engineer.

SPECIFICATIONS: AASHO Standard Specifications for Highway Bridges, current edition.

CONCRETE: Concrete in the precast prestressed piles and build-ups with driving shall have a minimum compressive cylinder strength (f'c) of 5000 psi at 28 days. Concrete in build-ups without driving shall have a minimum compressive cylinder strength (f'c) of 3000 psi. Compressive cylinder strength at transfer of prestressing force shall be not less than 4000 psi.

Higher concrete strengths may be used and advantage may be taken of such greater strength for handling and driving stresses and column loading, subject to approval of Engineer.

Air entrained concrete is recommended for use in piles which will be subjected to cycles of freezing and thawing and wetting and drying.

PRESTRESSING REINFORCEMENT: Seven wire stress relieved strand shall conform to the general requirements of ASTM Designation A416, and may be either regular or high strength, in accordance with strand manufacturer's published tables. Subject to the approval of the Engineer, prestressing may be increased as required for handling or driving by increasing the number or size of strands. In general the unit prestress after losses should not exceed 0.2 f'c, unless special conditions warrant and appropriate adjustment is made in allowable pile capacity. Broken wires within individual strands will be permitted up to 2% of the total number of wires in each pile, providing that there is not more than one broken wire per strand. Two or more broken wires per strand will be cause for replacement of the strand, even though the two broken wires are within the 2% limitation.

BUILD-UPS & SPLICES: Build-ups, precast or cast-in-place, may be used if specified or authorized by the Engineer. Two prestressed pile sections may be spliced by the use of dowels extending from the tip of the upper prestressed section into cored or drilled holes in the lower prestressed section. For hollow core or cylinder piles, this splice connection may be made in the walls of the pile or in a solid plug at the head and tip of the spliced sections. The dowels shall have an area equal to 1-1/2% of the gross cross-section of pile and shall be adequately bonded into both sections. The dowel holes and space between spliced sections shall be filled with a material having properties fully equal to that of the concrete and adhesive strength equal to the shear and tensile strength of the concrete. Such properties shall be obtained within a time limit consistent with the driving requirements of the pile.

Any alternate method of splicing providing equal results may be considered for approval.

CHAMFERS AND CORNERS: All corners of square piles shall be chamfered to at least 3/4" or rounded to approximately 1" radius.

FORMS: For forming the exterior of piles, the use of steel forms on concrete founded casting beds is required, unless otherwise approved by the Engineer. Side forms for square and octagonal piles, may have a maximum draft on each side not exceeding 1/4" per foot.

For For forming the interior of piles with hollow cores, forms shall be constructed of an approved material which will not deform or break during prestressing operations. If a moving mandrel is used for forming the inner void, special precautions shall be taken to prevent fallout of inner surfaces, tensile cracks and separation of concrete from strands.

PICK-UP AND HANDLING: Maximum lengths for pick-up are determined using the following stress assumptions.

Loading: 1-1/2 times full dead load. Allowable tensile stress equals $6.0\sqrt{f'c}$. These stress and loading criteria are based on normal care in handling the pile. If handling is such that damage to the pile becomes evident, the Engineer may require a higher load factor or lower allowable stress as necessary to insure no damage to piles.

DRIVING: Pile heads shall be protected from direct impact of the hammer by cushion blocks consisting of several plies of soft compressible wood or other approved material.

Jetting will be permitted and/or required when necessary to obtain the required penetration. Internal jets may be installed provided they are securely anchored to the pile and are imbedded in the concrete.

The driving head (helmet) shall be sufficiently large and shallow so as not to bind the head of the pile if it twists slightly during driving. Hollow piles which are open-ended at the tip shall have vents to relieve internal hydraulic pressure.

TOLERANCES: Voids, when used, shall be located within 1/2" of position shown on plans.

Pile ends shall be plane surfaces and perpendicular to axis of pile with a maximum tolerance of 1/8" per foot transversely.

The maximum sweep (deviation from straightness measured along two perpendicular faces of the pile, while not subject to bending forces) shall not exceed 1/8" in any 10' of its length.

GENERAL: When piles are ordered in accordance with this standard plan, the standard pile details shall be used. Alternate pile heads, pile tips, splices, build-ups or other alternates shall be used only if specified or authorized by the Engineer.

Where specific methods are indicated for achieving a result, other methods which will insure equal results may be considered for approval by the Engineer.

Small areas of honeycomb which are purely surface in nature extending to a depth of no more than one inch may be repaired in a manner satisfactory to the Engineer. Honeycomb extending to the plane of reinforcing will be cause for rejection.

For locations where severe freezing is likely, cylinder piles that would otherwise have water within the shells shall be filled. The filling in that portion of their height subject to freezing shall be of sound concrete. Special care shall be taken to insure the soundness and uniformity of the concrete and to eliminate all pockets or voids where segregation or lack of density in the concrete would permit the formation of pockets of free water.

PILE CAPACITIES AND LOADING

(a) Loads on Piles as Columns – The maximum compressive stress on prestressed concrete piles in excess of the effective prestress shall not exceed the following:

$$fpc = 1000 \text{ for } L/D = 0 \text{ to } 12$$

$$fpc = 1240 - 20\, L/D \text{ for } L/D = 12 \text{ to } 25$$

where L = effective length of pile
D = diameter or width of pile
fpc = allowable unit compressive stress in psi exclusive of effective prestress.

For piles considered hinged at both ends, "L" shall be taken as the actual length of pile. For piles considered hinged at one end and fully fixed at the other end, "L" shall be taken as 0.7 of the length between hinge and assumed location of fixity. For piles considered fully fixed at both ends, "L" shall be considered as 0.5 of the length between the two assumed fixed ends. When pile is designed as a cantilever, "L" shall be considered as 2 times the length of the cantilever.

Where it is necessary to use piles with an L/D greater than 25, they shall be thoroughly investigated for elastic stability using recognized formulae and applying a minimum factor of safety of two.

The above factor shall apply either for direct axial load or combination of axial load and bending. Where bending stresses occur, the maximum allowable tensile stress in concrete shall not exceed 250 psi.

The above factors are based upon a design concrete strength (f'c) of 5000 psi. For any higher values, these stresses may be increased in direct proportion to design concrete strengths.

If an effective prestress in excess of 0.2 f'c is used, the stresses as permitted above shall be reduced accordingly.

(b) Capacity in End Bearing of Prestressed Concrete Piles – The capacity of prestressed concrete piles in end bearing shall not exceed 0.2 f'c.

(c) Capacity of Prestressed Concrete Piles as Friction Piles – The capacity of prestressed concrete piles shall preferably be determined by loading tests. The capacity shall be limited to the lesser of either (a) above or the safe capacity resulting from test loading.

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COMMITTEE ON BRIDGES & STRUCTURES
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PRESTRESSED CONCRETE INSTITUTE

GENERAL NOTES
PRESTRESSED CONCRETE PILES

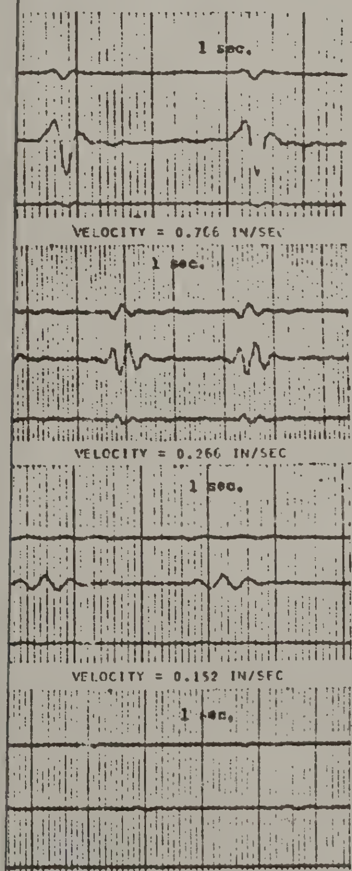
SUBMITTED BY

T. W. Jennings FOR AASHO

R. J. Lyman FOR PCI

Figure 1 shows the Sprengnether portable seismograph which we have used extensively, and an equivalent complement of electronic equipment and transducers. Typical recordings of vibration from pile-driving operations are shown in Figure 2.

The damage potential of pile-driving vibrations depends on the displacement and the frequency of the vibration. Neither of these two characteristics alone will damage a structure. Concerning displacement, it is common knowledge that a structure can be uniformly jacked through several feet without causing damage. Likewise, with regard to frequency, normal sound, in passing through a wall, can vibrate the wall at high frequencies (several thousand cycles per second) without



h vibrations from pile driving.

Damage Effects of Pile Driving Vibration

JOHN F. WISS, Wiss, Janney, Elstner and Assoc.

•PILE DRIVING, like dynamite blasts, nuclear blasts, and sonic booms, is a source of vibration which is frequently alleged to cause damage to structures. Unlike blasts, however, pile driving vibrations are produced by mechanical energy that is limited by the capabilities of the mechanical system. For example, a 5,000-lb ram falling freely from a height of 3 ft cannot deliver more than 15,000 ft-lb of energy on impact. Similarly, the maximum energy available from a double-acting steam hammer is limited by the steam pressure, the area of the piston, and the stroke.

On impact, the energy of the ram is imparted to the pile. It is distributed between rebound of the ram, elastic distortion of the pile, elastic and plastic deformation of the cushioning material, penetration of the pile, and elastic and plastic deformation of the earth surrounding the pile. The elastic deformation of the soil is propagated through the earth materials as elastic waves. The distribution of the available impact energy to the sources previously mentioned consists of interrelated functions, but the most important factor is the resistance of the soil to penetration by the pile. In a soft, easily penetrated soil, most of the energy is used in advancing the pile, and the least amount in the elastic deformation of the soil. In very hard, resistant soil the converse is true.

It is convenient to visualize the wave motion at the surface of the earth as being similar to the ripples produced on a smooth surface of water when a stone is thrown in. The wave length of the earth waves from pile driving is approximately 200 ft; this is the distance from the crest of one wave to the crest of the succeeding wave. Structures supported on the surface ride such waves in the same manner as a cork or box floating on the ripples of the water. Deeply embedded structures respond to a lesser degree in proportion to the orbital diameter of the earth particle motion which decreases exponentially with depth. For example, a structure embedded 200 ft below the surface would receive virtually no vibration. One at 100 ft would receive $^{1}/_{24}$th of the vibration experienced by a point on the surface. Regardless of depth, the magnitude of vibration intensity varies with the amount of energy transmitted to the soil, the physical properties of the soil, and the distance that the wave has traveled from the source.

Many instruments are capable of measuring the vibration intensities resulting from pile driving. Basically, such systems consist of a vibration sensor which converts the physical motion of the earth or structure into electrical signals. These in turn are sufficiently strengthened by an electronic amplifier to drive a galvanometer and produce a recording of vibration vs time. It is essential to record the vibration, because the impulses are transient, and the response of meters is not fast enough to follow the vibrations accurately. It is also important to record simultaneously the vibratory motion in three mutually perpendicular directions. Although the impact force is generally in the vertical direction, the maximum earth or structural vibration is not necessarily vertical.

The instrument most commonly used for measurement of earth or structural vibration resulting from pile driving is the portable three-component seismograph. This unit is a mechanical optical system which utilizes seismic principles, is portable and battery operated, and produces a recording of displacement in three mutually perpendicular directions vs time. It is ideally suited for field recordings of the vibrations associated with pile driving.

Paper sponsored by Committee on Construction Practices—Structures and presented at the 45th Annual Meeting.

13-7

Figure 1. Sprengnether portable seismograph and equivalent complement of electronic equipment and transducers.

Figure 1 shows the Sprengnether portable seismograph which we have used extensively, and an equivalent complement of electronic equipment and transducers. Typical recordings of vibration from pile-driving operations are shown in Figure 2.

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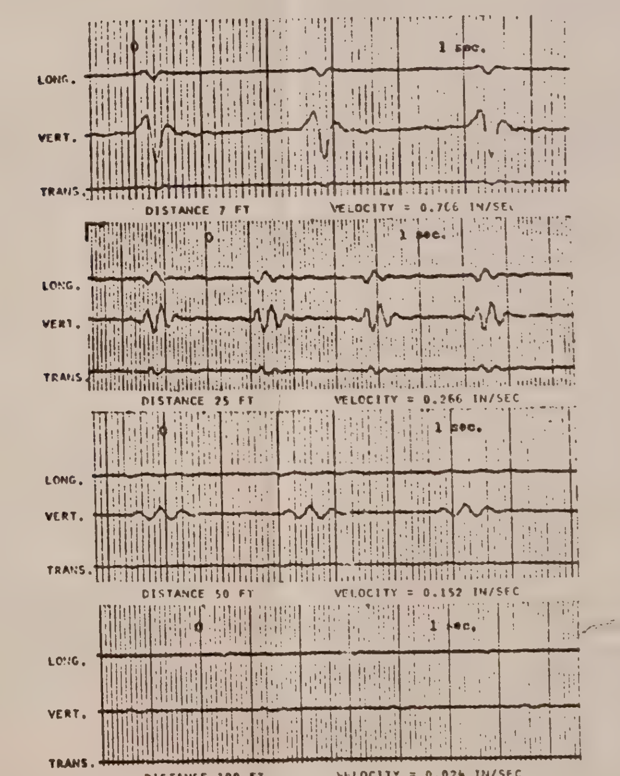
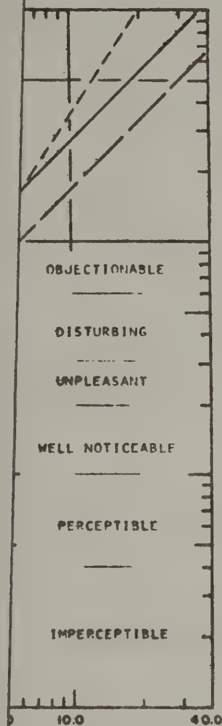


Figure 2. Typical earth vibrations from pile driving.

13-8

ENERGY



ving on wet sand, dry sand, and clay.

(amount of motion) and frequency
orne vibration is the best meas-
nent and frequency in the most
expressed as $2 \pi f A$, in which f
Impact vibrations produced by
on the type of soil. A loose
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ne investigators agree that a
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that ground motion particle ve-
ge for engineer structures.

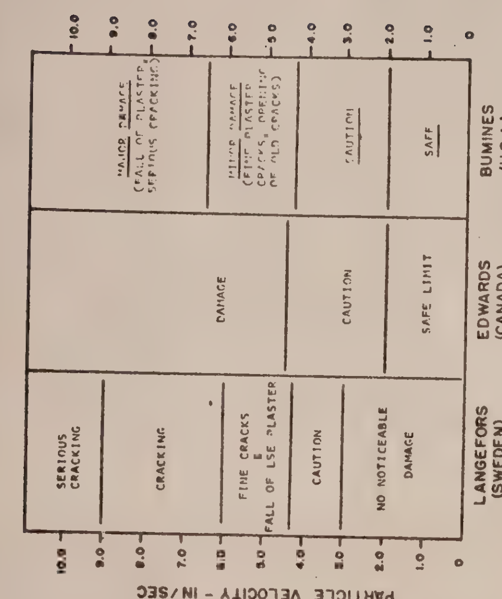


Figure 4. Comparison of damage criteria, residential-type structures.

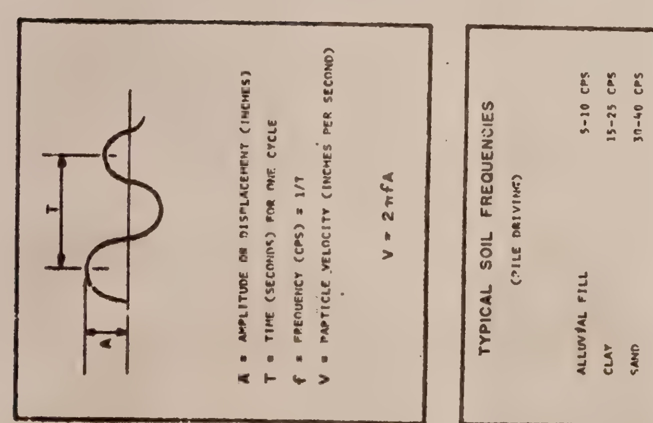


Figure 3. Particle velocity in alluvial fill, clay, and sand.

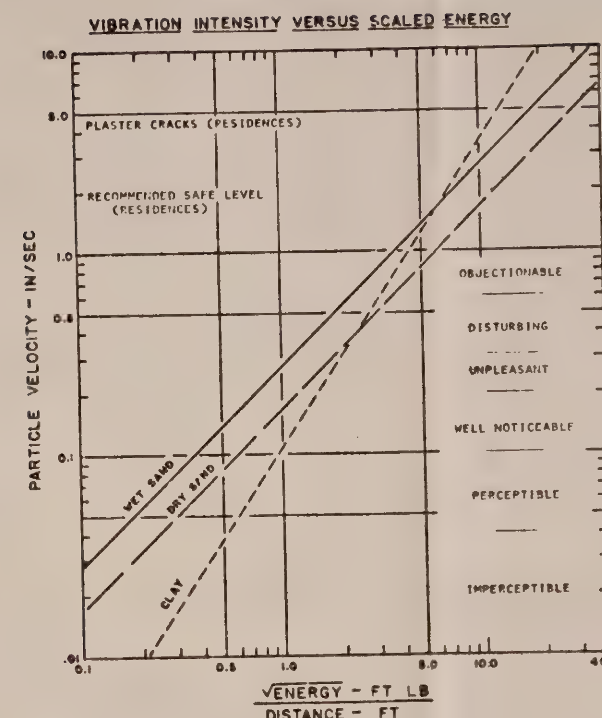
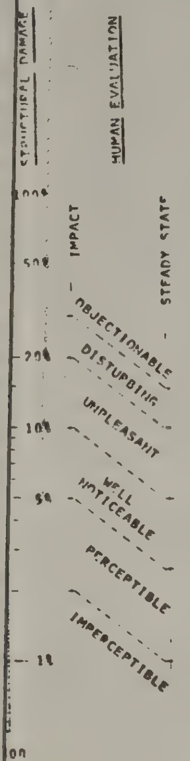


Figure 5. Maximum vibration intensities expected from pile driving on wet sand, dry sand, and clay.

causing damage. It is a combination of displacement (amount of motion) and frequency which causes damage. The particle velocity of earthborne vibration is the best measure of damage potential because it combines displacement and frequency in the most significant manner. Particle velocity (Fig. 3) can be expressed as $2\pi fA$, in which f is frequency (cps) and A is amplitude (displacement). Impact vibrations produced by pile driving have characteristic frequencies depending on the type of soil. A loose alluvial fill has natural frequencies of about 5 to 10 cps, clay soils vary between 15 and 25 cps, sand between 30 and 40 cps.

Several investigators in this country and abroad (including the U.S. Bureau of Mines) have found that particle velocities in excess of 4.0 in./sec are required to cause plaster cracks in dwellings. Figure 4 shows a comparison of the results of several of the investigations. With appropriate conservatism, the investigators agree that a vibration level of 2.0 in./sec (particle velocity) is safe with regard to plaster cracks in residential-type structures.

The effect of ground motion on an engineered structure can be computed by commonly used methods in the earthquake engineering field. The structure is considered a lumped mass-spring dashpot system, and its response to a series of impacts can be calculated. Based on observation and experience, it can be stated that ground motion particle velocities below 4.0 in./sec are well within the safe range for engineer structures.



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project involves the case in
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ncrete technology, it is doubt-

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en concrete attains strength,
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concrete (28-day strength)
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on are, therefore, 0.25 and
, these vibration particle ve-
(Fig. 5). If a 15,000-ft-lb

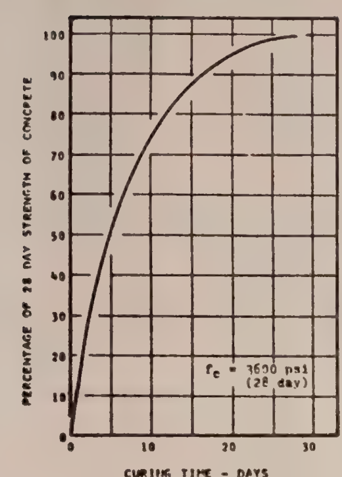


Figure 6. Strength of concrete vs curing time.

Figure 5 shows the maximum vibration intensities to be expected from pile driving in several soils on which extensive data have been obtained by the autho . The data are plotted on log-log paper in which the abscissa is $\sqrt{E/D}$. This scaled energy factor permits use of the graphs with any size of pile driver; E is the foot-pounds of energy delivered by the hammer, and D is the seismic distance, in feet, from the pile tip to the location of interest. The vibration intensity (particle velocity) varies as the square root of the energy of the hammer. Figure 5 also indicates the levels at which vibration damage may be expected and the normal human evaluation of pile driving vibration. In several investigations, vibrations resulting from the driving of sheet piling, wood piles, and H piles were measured. For all practical purposes there is no difference in the vibration produced, all other variables being constant.

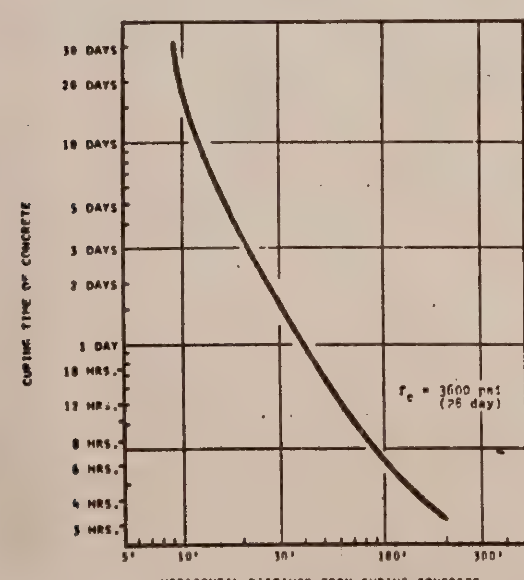


Figure 7. Limiting safe distance vs curing time of concrete for pile driver rated at 15,000 ft-lb of energy.

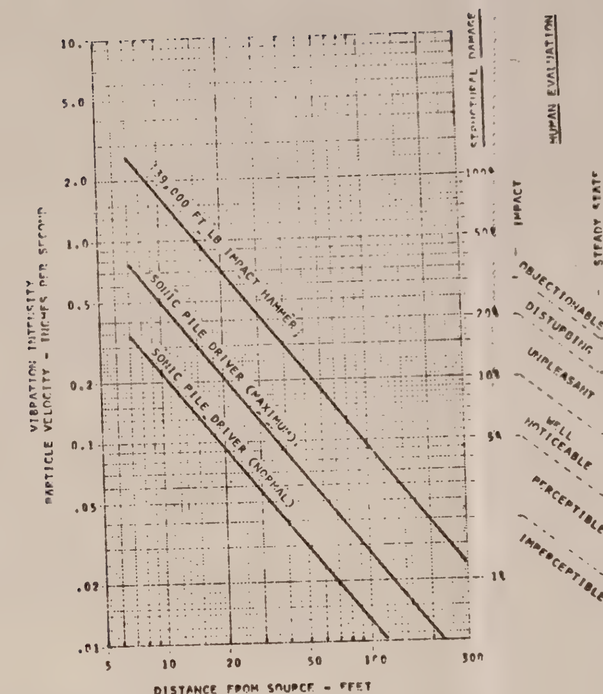


Figure 8. Vibration intensity vs distance.

Another problem of common interest on a construction project involves the case in which piles are to be driven at the same time that concrete is being placed. The question has frequently been raised as to whether the pile driving might have a detrimental effect on "green" concrete. Until more is known about concrete technology, it is doubtful that a rigorous analysis of such effects can be made.

As a practical matter, the following reasoning has been used by the author in the past, and although the magnitude of the safety factor has not been determined, there has been no evidence to indicate that the approach is not conservative.

Assuming that 5.0 in./sec (particle velocity) is conservatively a safe vibration level for cured concrete, and recognizing the rate at which green concrete attains strength, it is then possible to relate the permissible safe vibration level to the time since placing the concrete. When the decrease of vibration with distance has been evaluated for a particular site, the distance at which pile driving may be permitted (for a certain size hammer) can then be determined as a function of curing time.

Assume, for purposes of illustration, that a 3,600-psi concrete (28-day strength) has been specified on a particular project. The percentage of 28-day strength of concrete vs curing time is shown in Figure 6. This curve shows that the concrete has approximately 5 percent of its strength in 12 hr (one-half day), or 10 percent in 24 hr (one day). Thus, the vibration intensity should not exceed this same percentage of a particle velocity of 5.0 in./sec; limiting values of vibration are, therefore, 0.25 and 0.5 in./sec, respectively. If the soil is basically a clay, these vibration particle velocities correspond to an $\sqrt{E/D}$ of 2 and 3, respectively (Fig. 5). If a 15,000-ft-lb

pile driver is used, the closest permissible distances are 61.5 and 41.0 ft, respectively.

By the foregoing method a curve can be developed in which the limiting safe distance for pile driving vs curing time of concrete can be determined (Fig. 7). This curve is represented as a typical evaluation. For a specific site, pile driver, and concrete, limiting distances and curing time should be based on measured vibration intensities and concrete strength determinations, especially where short curing times are involved.

In closing, some brief comments on the vibration produced by the sonic pile driver and by other vibratory pile drivers are pertinent. In contrast to the impact hammer, which excites the soil at its natural frequency and the vibrations die out before the next blow, the sonic and vibratory pile drivers force the soil to vibrate at the continuous frequency (rpm) of the driver. These units can be speed controlled over a limited frequency range. Investigations of a sonic pile driver driven at frequencies between 90 and 120 cps, and a vibratory pile driver adjustable between 16 and 21 cps resulted in the following observations.

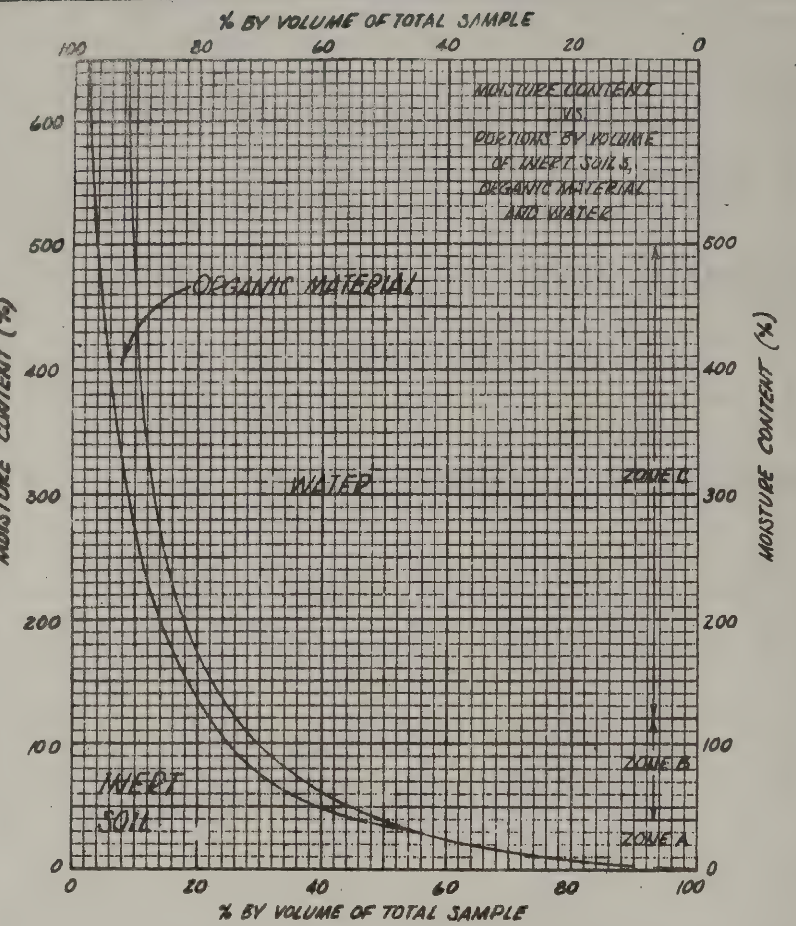
The normal vibration levels from the sonic driver may by one order of magnitude lower than those of an impact pile driver. However, the vibration varies continuously and occasionally attains intensities approximately one-half of the levels produced by a comparable impact hammer. Further, because the vibration is of a steady-state rather than a transient character, the human evaluation is usually more pronounced—by a factor of 2 (Fig. 8). The other vibratory pile driver investigated produced vibration levels of the same order of magnitude as a comparable impact pile driver.

With a steady-state excitation the possibility of resonance response in building components (especially panels) may become of significant importance. In the case of the transient vibrations produced by an impact pile driver, the duration of the transient is sufficiently short (0.2-0.3 sec) that a resonance buildup of structural components is not likely. The safe level of intensity for a steady-state vibration could conceivably be between one-half and one-fifth of the safe level for transient excitation; this is due to the possible magnification at resonance which depends primarily on the inherent damping characteristics of the structure.

SECTION 17

ORGANIC SOILS

PAGES



ENGINEERING CHARACTERISTICS OF ORGANIC SOILS

VISUAL IDENTIFICATION AND MOISTURE CONTENT ARE TWO SIMPLE TESTS USED TO ESTIMATE THE ENGINEERING PROPERTIES OF ORGANIC SOILS.

ON THE ADJACENT PLOTS THE SOILS HAVE BEEN PLACED IN THREE GENERAL CATEGORIES:

ZONE A – FINE SANDS AND SILTS WITH SMALL AMOUNTS OF ORGANIC – USUALLY OFFER NO CONSTRUCTION PROBLEMS.

ZONE B – INERT SOIL MIXED WITH ORGANIC MATERIAL – MAY BE STABILIZED IN SOME CASES.

ZONE C – ORGANIC SOIL WITH SMALL AMOUNTS OF INERT – USUALLY UNSUITABLE FOR HIGHWAY FOUNDATION AND MUST BE REMOVED BY EXCAVATION OR DISPLACEMENT.

EXAMPLE SHOWING USE OF CHART:

AT MOISTURE CONTENT OF 200% A SATURATED SOIL CONTAINS BY VOLUME 14% INERT SOIL, 4% ORGANIC MATERIAL AND 82% WATER.

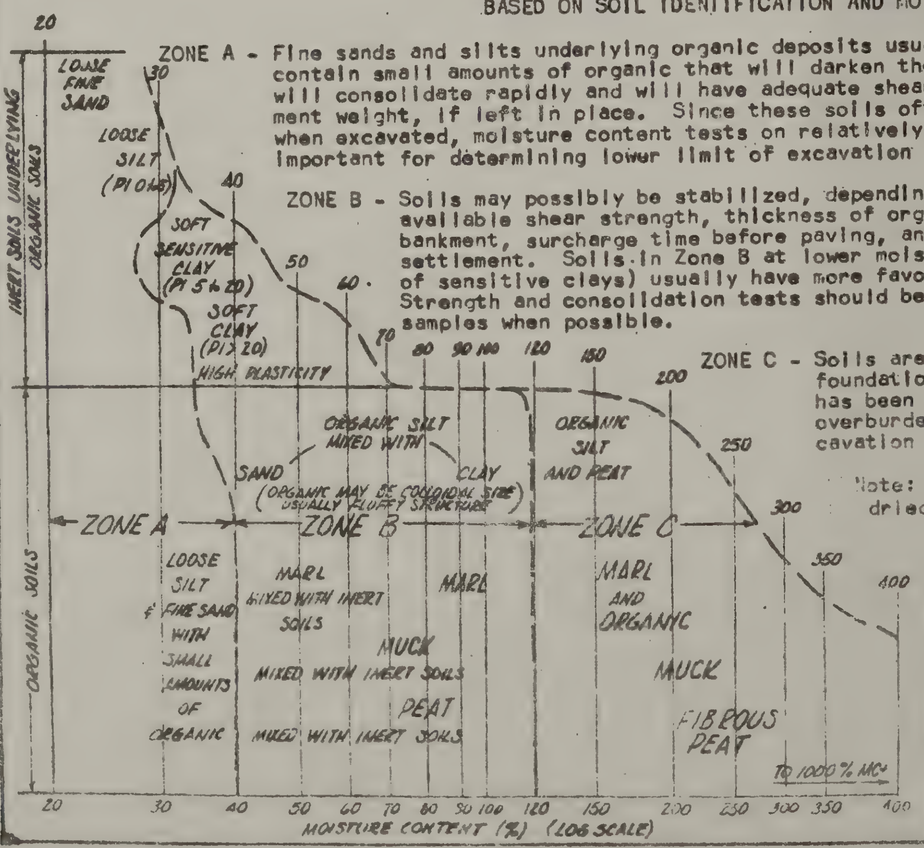
PRELIMINARY CHART FOR ESTIMATING SUITABILITY OF ORGANIC SOILS FOR HIGHWAY EMBANKMENT FOUNDATIONS BASED ON SOIL IDENTIFICATION AND MOISTURE CONTENT

ZONE A – Fine sands and silts underlying organic deposits usually are loose and sometimes contain small amounts of organic that will darken the soil color. These soils will consolidate rapidly and will have adequate shear strength to support embankment weight, if left in place. Since these soils often appear to be "unsuitable" when excavated, moisture content tests on relatively undisturbed samples will be important for determining lower limit of excavation in field.

ZONE B – Soils may possibly be stabilized, depending on rate of consolidation, available shear strength, thickness of organic deposit, height of embankment, surcharge time before paving, and allowable post-construction settlement. Soils in Zone B at lower moisture contents (with exception of sensitive clays) usually have more favorable engineering properties. Strength and consolidation tests should be conducted on undisturbed samples when possible.

ZONE C – Soils are unsuitable for embankment foundations unless deposit is thin or has been precompressed by existing overburden. Otherwise remove by excavation or displacement.

Note: A visual inspection of the dried sample will aid in identification of soil.



PRELIMINARY

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

ENGINEERING PROPERTIES OF ORGANIC SOILS

APPROVED 19	DISTRICT NO. COUNTY
PRINCIPAL SOILS ENGINEER	DWG. NO. SM 16 [illegible]

SECTION 20

SPECIAL SPECIFICATIONS

SPECIAL SPECIFICATION

ITEM 17203.1399 PIEZOMETERS (Air Activated)

Description

This work shall consist of furnishing, installing and maintaining air activated piezometers and appurtenances at the locations and elelvations designated in the contract documents or as ordered by the Engineer.

Materials

1. Air activated pressure cells shall be capable of operating within a range of 1 to 100 pounds per square inch and shall be sensitive to .05 pounds per square inch. The maximum allowable dimensions shall be 1-3/4 inches in diameter and 12 inches in length.

2. The readout device shall be equipped with an air pressure gauge(s) capable of reading a maximum pressure based on the depth of installation plus the surcharge weight of added embankment material. Gauges shall have the following accuracy:

Range (PSI)	Accuracy (PSI)
0 - 15	± .75
0 - 30	± 1.50
0 - 60	± 3.0

3. Two air leads of nylon pressure tubing, 3/16 inch O.D., shall be provided in such lengths that no connections shall be made in the area under the embankment except at the cell or as ordered by the Engineer. All connectors shall be brass "Imperial flex unions," or equal. Only new, unused tubing will be acceptable for use on this contract.

4. The tamping hammer shall weigh 30 pounds ± 10% with a maximum diameter of two inches. The hammer can be either split barrel or one piece but shall conform to the dimensions shown on the plans.

5. A 1/8 inch diameter galvanized preformed airplane cable of sufficient length to permit installation of the deepest piezometers shall be supplied and have a snap-type swivel hook attached to one end.

6. A tripod and sheave shall be supplied for operating the tamping hammer.

7. Drive sample drilling equipment shall be supplied to obtain soil samples.

8. The sand used for piezometer installation shall be Ottawa sand, or another thoroughly washed sand between No. 20 and No. 40 mesh in grain size.

Materials (Cont.)

9. The seals shall be made of 1/2 inch diameter "Pi Pellets" which can be obtained from Piezometer Research & Development Corporation, 33 Magee Avenue, Stamford, Connecticut 06902.

10. Rounded gravel approximately 1/2 inch in diameter shall be used on top of each bentonite seal.

11. Gauge protection boxes shall be constructed as shown on the plans or in the proposal.

Construction Details

The Contractor shall utilize personnel experienced with the installation of piezometers. Prior to commencing the piezometer installation, the Contractor shall provide to the Engineer for his approval, a description of the applicable previous experience of such personnel.

All phases of the installation of piezometers, including connection of the system, are to be inspected by a representative from the Soil Mechanics Bureau. Piezometers will be accepted for conformance with the specification requirements on the basis of this inspection.

A. No embankment material shall be placed within 100 feet of the proposed location of a piezometer until the piezometer has been installed and all connections completed in conformance with the following procedures:

   1. At the location shown on the plans, a 2-1/2 inch diameter casing shall be driven to the approximate elevation of the bottom of the piezometer cell. The bottom 10 feet of casing must be in one piece, without joints or couplings, and shall not have a drive shoe on the lower end. The casing may be advanced by any means, except for the final twenty feet of penetration. It shall then be driven in five foot increments. The casing must be washed out after each five foot advance. The casing shall be kept filled with water at all times. No washing below the casing will be permitted.

      The upper section or sections of casing shall be tightened to a lesser degree than the lower sections of casing to facilitate disconnection of the casing in Step 11 below.

   2. A split spoon sample shall be obtained of the material within a depth of 12 inches below the bottom of the casing and delivered in sealed jars to the Engineer. The casing shall be driven to 12 inches below the piezometer cell elevation and the remaining soil cleaned out to the bottom of the casing.

   3. The casing shall be pulled up one foot and approved sand poured into the casing to fill up the one foot void. The top of the sand shall be at the bottom of the casing and measured with the tamping hammer.

Construction Details (Cont.)

4. The air leads shall be connected to the air piezometer cell. The system shall be checked for leaks and the tubing labeled or color coded as to intake and exhaust before installation. The cell assembly shall be lowered into the casing and readings taken at convenient depths to confirm the cell accuracy until the cell has reached the planned elevation. The tamping hammer shall be lowered over the tubing to the top of the cell to center the cell in the hole.

5. The hammer shall be removed and a measured volume of sand poured into the casing so that the sand fills the space around the piezometer tip and to approximately 2-1/2 feet above the bottom of the casing. Tension shall be maintained on the tubing but vertical movements of the piezometer tip shall not be permitted.

6. A one inch thick layer of 1/2 inch diameter gravel shall be formed on top of the sand in the casing and tamped with 20 blows of the hammer falling six inches.

7. While maintaining a constant tension on the tubing, a bentonite seal with five separate layers shall be formed. Each layer shall consist of a three inch thickness of Pi Pellets and a one inch thickness of gravel. Each layer shall be placed and compacted as follows:

    a. The water shall be lowered three inches below the top of the casing. If the casing cannot be kept full of water, another method of measuring the necessary amount of Pi Pellets shall be proposed by the Contractor and approved by the Engineer.

    b. Pi Pellets shall be dropped individually into the casing until the water rises to the top of the casing and sufficient time allowed for the balls to reach the bottom (about 1 minute for each 10 feet of depth).

    c. The water shall be lowered one inch below the top of the casing. If the casing cannot be kept full of water, another method of measuring the necessary amount of rounded gravel shall be proposed by the Contractor and approved by the Engineer.

    d. One half inch diameter gravel shall be dropped into the casing until the water rises to the top of the casing. Sufficient time shall be allowed for the gravel to reach the bottom (about 1 minute for each 10 feet of depth).

Construction Details (Cont.)

e. The tamping hammer shall be slipped over the tubing and, keeping tension on the tubing, 20 blows applied to the gravel layer with a six inch drop of the hammer. Whenever the tamper does not move freely, it shall be immediately withdrawn and cleaned.

f. This procedure shall be repeated until a five-layer seal is formed.

8. Enough sand shall be poured into the casing to form a two foot layer. It shall be covered with a one inch layer of 1/2 inch diameter gravel and tamped with 20 blows of the hammer falling six inches.

9. Step 7 shall be repeated forming another five-layer seal.

10. The remainder of the casing shall be filled with sand.

11. The top section of the casing shall be disconnected so that the top of the casing is at least four feet below the ground surface.

12. The horizontal air leads shall be laid zig-zag in a minimum one foot wide and two feet deep trench to prevent disturbance of the lines by any subsequent grading operation. The air leads shall not cross in the trench.

13. The horizontal air leads shall be connected to the gauge protection box. Each lead shall then be fully labeled.

B. A piezometer will be maintained until the Director of the Soil Mechanics Bureau determines that the installation may be abandoned. When a piezometer becomes damaged or inoperable, all earthwork operations shall be stopped within 100 feet of the installation until the installation is either repaired or abandoned. The Director of the Soil Mechanics Bureau will determine if replacement is required or if sufficient information has been obtained so that the installation may be abandoned. Piezometers that become damaged or became inoperable through no fault of the Contractor shall be replaced and paid for at the unit price bid for this item. Any piezometer that becomes damaged or inoperable as a result of the Contractor's operations will be replaced by him at no cost to the State.

Method of Measurement

The quantity will be the actual number of piezometers satisfactorily installed in accordance with this specification.

Basis of Payment

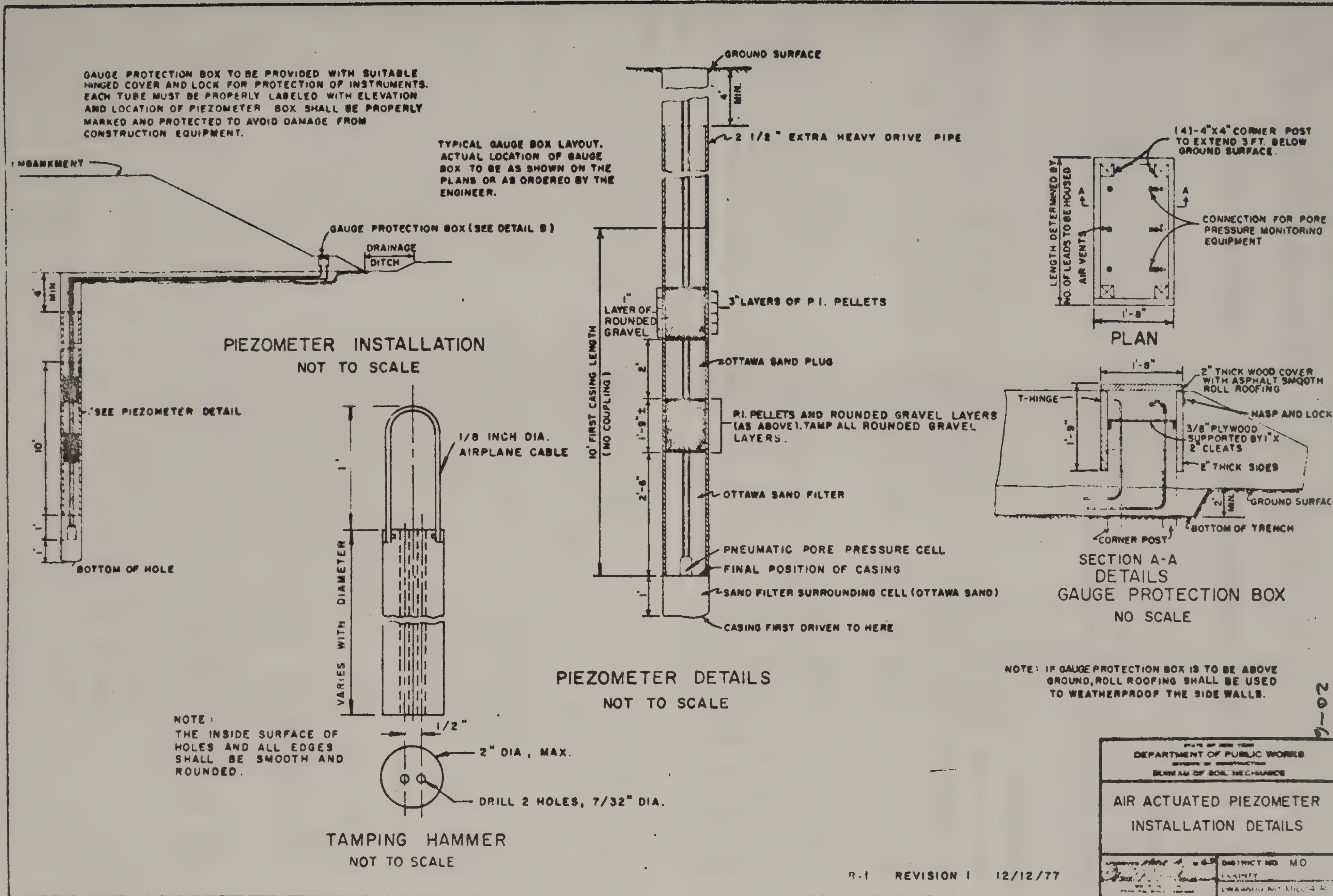
The unit price bid for this Item shall include the cost of furnishing all labor,

Basis of Payment (Cont.)

material and equipment necessary to complete the work, including the installation and maintenance of the installation. In addition, at the completion of the contract, the piezometers and appurtenances, including readout devices, shall become the property of the State.

No separate payment will be made for the cost of excavation and backfill of the trench to install the horizontal air leads. The cost of this work will be included in the price bid for this Item.

No payment will be made for any piezometer that becomes damaged or inoperable as a result of the Contractor's operations.


GAUGE PROTECTION BOX TO BE PROVIDED WITH SUITABLE HINGED COVER AND LOCK FOR PROTECTION OF INSTRUMENTS. EACH TUBE MUST BE PROPERLY LABELED WITH ELEVATION AND LOCATION OF PIEZOMETER BOX SHALL BE PROPERLY MARKED AND PROTECTED TO AVOID DAMAGE FROM CONSTRUCTION EQUIPMENT.
EMBANKMENT
TYPICAL GAUGE BOX LAYOUT. ACTUAL LOCATION OF GAUGE BOX TO BE AS SHOWN ON THE PLANS OR AS ORDERED BY THE ENGINEER.
GAUGE PROTECTION BOX (SEE DETAIL B)
DRAINAGE DITCH
4' MIN
10'
1'
1'
SEE PIEZOMETER DETAIL
BOTTOM OF HOLE
PIEZOMETER INSTALLATION
NOT TO SCALE
1'
1/8 INCH DIA. AIRPLANE CABLE
VARIES WITH DIAMETER
NOTE: THE INSIDE SURFACE OF HOLES AND ALL EDGES SHALL BE SMOOTH AND ROUNDED.
1/2"
2" DIA, MAX.
DRILL 2 HOLES, 7/32" DIA.
TAMPING HAMMER
NOT TO SCALE
GROUND SURFACE
4' MIN.
2 1/2" EXTRA HEAVY DRIVE PIPE
1" LAYER OF ROUNDED GRAVEL
3" LAYERS OF P.I. PELLETS
OTTAWA SAND PLUG
10' FIRST CASING LENGTH (NO COUPLING)
2'
1'-9"±
2'-6"
1'
P.I. PELLETS AND ROUNDED GRAVEL LAYERS (AS ABOVE). TAMP ALL ROUNDED GRAVEL LAYERS.
OTTAWA SAND FILTER
PNEUMATIC PORE PRESSURE CELL
FINAL POSITION OF CASING
SAND FILTER SURROUNDING CELL (OTTAWA SAND)
CASING FIRST DRIVEN TO HERE
PIEZOMETER DETAILS
NOT TO SCALE
(4)-4"X4" CORNER POST TO EXTEND 3 FT. BELOW GROUND SURFACE.
LENGTH DETERMINED BY NO. OF LEADS TO BE HOUSED
AIR VENTS
A
A
CONNECTION FOR PORE PRESSURE MONITORING EQUIPMENT
1'-8"
PLAN
1'-8"
2" THICK WOOD COVER WITH ASPHALT SMOOTH ROLL ROOFING
T-HINGE
HASP AND LOCK
1'-9"
3/8" PLYWOOD SUPPORTED BY 1" X 2" CLEATS
2" THICK SIDES
2' MIN.
GROUND SURFACE
BOTTOM OF TRENCH
CORNER POST
SECTION A-A
DETAILS
GAUGE PROTECTION BOX
NO SCALE
NOTE: IF GAUGE PROTECTION BOX IS TO BE ABOVE GROUND, ROLL ROOFING SHALL BE USED TO WEATHERPROOF THE SIDE WALLS.
20-6
DEPARTMENT OF PUBLIC WORKS
AIR ACTUATED PIEZOMETER
INSTALLATION DETAILS
DISTRICT NO. MO
R-1 REVISION 1 12/12/77

ITEM 17 203.0302 EMBANKMENT IN PLACE (Lightweight Fill)

Description

Under this Item, the Contractor shall furnish and place lightweight fill necessary to complete the embankments shown on the plans or as ordered by the Engineer.

Materials

The material shall be blast furnace slag, expanded shale or other materials as approved by the Deputy Chief Engineer, Technical Services. The material shall have a maximum particle size of 24 inches in greatest dimension and the compacted wet density shall not exceed the density specified in the proposal as measured in a test embankment.

Construction Details

The compacted wet density shall be determined in test embankments containing a minimum of 400 cubic yards of material constructed on firm flat surfaces. The Contractor shall construct each test embankment in an area bounded by 100 ft. by 50 ft. dimensions and he shall give the Engineer at least (1) one week written notice prior to beginning each test in order for the location to be inspected and surveyed.

The lightweight fill material shall be stored in piles not exceeding 20,000 cubic yards prior to testing. Representative material from each storage pile shall be used to construct a test embankment to a minimum height of four (4) feet in accordance with this specification.

The Contractor shall weigh all the material prior to placement in the test embankment. The embankment shall be constructed in uniform layers not exceeding 24 inches in thickness prior to compaction. Each layer shall be rolled over its entire area by a vibratory steel drum roller. The number of passes, the size of vibratory steel drum roller, and the need for actually vibrating the roller will be as directed by the Engineer.

The Engineer shall determine the volume of the test embankment. If the compacted wet density of the material in the test embankment is greater than the specified density, both the material contained in the test embankment and the material from the storage pile it represents shall be rejected for use under this Item.

The design embankments shall be constructed using the same methods, equipment and procedures used to construct the test embankments. However, the following requirements contained in Section 203 shall not apply:

a. The density requirements both in the embankment and in the subgrade area.

b. The maximum particle size in the subgrade area.

c. Proofrolling.

ITEM 17203.0302 EMBANKMENT IN PLACE (Lightweight Fill)

The top surface of the lightweight embankment lying directly beneath the subbase course materials shall be chinked to the satisfaction of the Engineer with lightweight material to prevent infiltration of the subbase materials.

Method of Measurement

The quantity of lightweight fill to be paid for under this Item shall be the number of cubic yards of material computed in its final compacted position between the payment lines shown on the plans or between revised payment lines established by the Engineer prior to performing the work.

Basis of Payment

The unit price bid per cubic yard shall include the cost of furnishing all labor, material and equipment necessary to complete the work including the test embankments.

No payment will be made for any loss of material which may result from foundation settlement, erosion or any other cause. The cost of such losses shall be included in the price bid for this Item.

ITEM 17605.3101 VERTICAL SAND DRAINS

Description

Under this Item the Contractor shall furnish all necessary equipment and materials to install vertical sand drains by the continuous hollow shaft auger method or the jetted in casing method or an equal minimum displacement procedure subject to the approval of the Director of the Soil Mechanics Bureau. All work shall be performed in accordance with the details shown on the plans and specifications.

The Contractor's attention is directed to the existence of U. S. Patent No. 3,096,622, issued July 9, 1963, to Mr. Richard A. Landau, which covers the continuous hollow shaft auger method of sand drain installation.

Materials

The sand used for backfilling the sand drains shall not contain any organic or other deleterious materials and shall meet the following gradation requirements:

Sieve Size	Percent Passing by Weight
3/4 inch	100
3/8 inch	80-100
No. 10	25-100
No. 40	0-35
No. 100	0-8
No. 200	0-2

The material shall be stockpiled and tested in accordance with the procedures contained in the appropriate Departmental publications in effect on the letting date of the project.

Construction Details

Vertical sand drains shall be installed at the locations shown on the plan after the initial 3 foot thick uncompacted drainage blanket (working platform) is installed. The sand drains shall be installed using a minimum 18 inch outside diameter hollow shaft auger, a minimum 12 inch outside diameter jetted in casing, or an approved equal. They shall be progressed through the working platform and underlying peat deposit and into the underlying clay deposit to elevation + 352 feet. Under no circumstances shall the sand drains be progressed below elevation + 352 feet due to the presence of an artesian pressure below this elevation. The drains shall then be backfilled with sand meeting the requirements specified under "Materials."

The drains shall be spaced and arranged as shown on the plans unless otherwise directed by the Engineer.

If obstructions are encountered and cannot be penetrated by the sand drain apparatus, the Contractor may abandon the hole and install the sand drain at a location specified by the Engineer. All abandoned holes must be backfilled in a manner subject to the approval of the Engineer to insure the support of the surrounding soil.

ITEM 17605.3101 VERTICAL SAND DRAINS

The Contractor shall provide the Engineer with a suitable means of making a volumetric determination of the quantity of the sand used to backfill each drain. If it is determined by the Engineer that the quantity of sand used is less than the theoretical volume of the hole, the Contractor shall immediately ascertain the reason and adjust or change his operations to correct any deficiency prior to the continuation of sand drain installations.

Vertical sand drains that are not in their proper location, or drains that are damaged in forming or during the placing of sand backfill, or drains not completed to the satisfaction of the Engineer, shall be rejected and replaced with drains that meet the criteria of the specifications.

Where instrumentation such as piezometers and settlement platforms are to be installed, the location of completed vertical sand drains shall be marked with stakes to insure that the instrumentation is accurately placed between sand drains.

The Contractor shall be prepared to install up to five trial drains in order to establish a satisfactory rate of penetration and/or flow rate for production drains in representative areas. Such tests shall be performed under the direction of the Engineer. The procedure thus established will then be used for all remaining drains.

Installation by the Continuous Hollow Shaft Auger Method

1. The outside diameter of the auger shall be a minimum 18 inch diameter and shall not be greater than sixty percent of the outside diameter of the helix. The inside diameter of the hollow shaft shall be greater than six inches. The auger shall be straight and of constant diameter.

2. The rate of advance of the auger shall not be greater than one pitch length per revolution and shall be adjusted so that the volume removed by the helix at any depth is equal to the volume displaced by the hollow shaft at that depth. Suitable means shall be provided such that soil does not enter the bottom of the shaft during the advancement of the auger. Suitable means shall also be provided to retard the advance of the auger in areas of very soft soil, which might otherwise result in the unit "plunging" under its own weight and displacing soil.

3. At the end of the auger advance for each sand drain, the auger shall be held stationary in the vertical position and rotated at least one revolution. The sand shall then be placed in the hollow shaft and the auger withdrawn in such a manner that a continuous column of sand extends from the bottom of the drain to the top surface of the working platform.

4. The rate of auger withdrawal shall be regulated so that the sides of the hole are supported at all times, either by the sand backfill or the soil in the auger helix. The auger shall not be rotated during extraction without the written permission of the Engineer.

ITEM 17605.3101 VERTICAL SAND DRAINS

5. The material taken from holes shall be considered unsuitable material and removed and disposed of in accordance with Section 203.3.08 "Disposal of Surplus Excavated Materials."

Installation by the Jetted in Casing Method

1. In order to minimize displacement and remolding of soils adjacent to the 12 inch minimum diameter sand drain the jetting tool must be advanced in such a manner that all soils which are penetrated are removed by the jetting and washing action. The pulsating bailor or "Dutch Bailor Method" will be acceptable. The rate of advance of the jetting tool and the flow rate of jetting water shall be controlled so that the unit does not advance faster than the soil is removed by the jetting action. In no case shall the rate of advance be greater than 20 feet per minute.

2. The water pressure and volume of water must be such to guarantee satisfactory progress of sand drain installation. When the hole has reached the required tip elevation, the jetting shall be continued and/or water shall be pumped until the water in the hole contains not more than a total of 2 percent by weight of silt and clay size particles.

3. During progression of the hole, the Contractor shall provide a suitable means of determining the depth of the hole at any given time.

4. Each hole shall be inspected and approved by the Engineer before the sand is placed therein to insure that the jetting action has not resulted in the removal of soil below the designed depth of the drains.

5. The sand backfill material shall be placed as the casing is withdrawn by a method that will produce a continuous column of approved clean sand for the full diameter and depth of the vertical sand drain.

6. Jetted material shall not be permitted to come in direct contact with the free draining material making up the working platform in the sand drain area and for a distance of 50 feet from the area of the sand drains. The Contractor's attention is directed to the fact that no wash water from the jetting method shall be put into streams directly or indirectly without special treatment to avoid pollution. The Contractor shall have his own arrangements for disposing of such material. The Contractor shall submit to the Engineer for his approval a plan for temporary pollution control for the selected method of sand drain installation prior to beginning the work.

Method of Measurement

The quantity of vertical sand drains will be the actual total length, in linear feet of sand drains installed and accepted, measured from the top of the working platform to the bottom of the sand drains (elevation + 352 feet) as directed or approved by the Engineer. There will be no measurement for payment for rejected or abandoned holes.

ITEM 17605.3101 VERTICAL SAND DRAINS

Basis of Payment

The unit price bid shall include the cost of forming the sand drain hole, filling the hole, disposal of material removed from the hole and shall also include the cost of furnishing all tools, materials, labor, equipment and all other costs necessary to produce the required end result in accordance with the plans and specifications. No direct payment will be made for trial drains or abandoned holes. The cost of temporary pollution control for work under this Item shall be included in the unit price bid for this Item.

ITEM__________COLLECTOR DRAINS

DESCRIPTION

Under this Item, the Contractor shall excavate and backfill collector drains as shown on the plans or as ordered by the Engineer.

MATERIALS

The material to be used to backfill the collector drains shall conform to the requirements of Section 703-02, Coarse Aggregate, and shall be primary size #1.

CONSTRUCTION DETAILS

The collector drains shall be constructed according to the plans and in a manner approved by the Engineer. Care shall be taken by the Contractor not to contaminate the sand drains during collector drain construction. Any sand drain material that is contaminated shall be removed and replaced at no cost to the State. As soon as the collector drains are installed, the Contractor shall cover the area with a 12 inch lift of embankment material to prevent contamination of the sand drains and collector drains. Any collector drain material that becomes contaminated by any means before the area is covered, shall be removed and replaced at no cost to the State.

METHOD OF MEASUREMENT

The quantity of collector drains will be the actual measured total length, in linear feet, of collector drains completed in accordance with the specifications.

BASIS OF PAYMENT

The unit bid price per linear foot shall include the cost of furnishing all labor, material and equipment necessary to complete the work. No direct payment will be made for any losses of material which may result from contamination, settlement, erosion or any other causes; the cost of such losses shall be included in the bid price for this item.

The cost of removing and replacing sand drain material to provide a continuous collector drain shall be included in the price bid for collector drains.

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ITEM 17203.98 SLOPE INDICATOR CASING

1.1 Description

The work shall consist of furnishing, installing and maintaining plastic slope indicator (S.I.) casing at the locations and depths specified in the contract documents or as directed by the Engineer (see Fig. 1 - Typical Slope Indicator Installation). If the slope indicator casing is damaged due to the Contractor's construction operations, work within 30 feet of the slope indicator shall be stopped and the slope indicator casing repaired or replaced at the Contractor's expense. At the completion of the contract, the slope indicator casing shall be in proper working order.

1.2 Materials

Slope Indicator

a. 2.75 inch O.D. plastic slope indicator casing (A.B.S. Plastic)
b. plastic couplings (A.B.S. Plastic)
c. protective caps (2 per hole)
d. assembly tools

The slope indicator materials shall be from the Slope Indicator Company (Sinco) or an equal approved in writing by the Director of the Soil Mechanics Bureau.

Grout Equipment

a. 1 inch diameter rigid plastic grout pipe - (P.V.C. Plastic, schedule 21)
b. plastic couplings (P.V.C. Plastic, schedule 21)
c. grout check valve according to Fig. 2 - Typical Grouting Check Valve Detail or equivalent
d. quick disconnect valve
e. Portland cement - Type 2 per NYSDOT Standard Construction Specifications, subsection 701-01
f. grout sand (if required to thicken grout) per NYSDOT Standard Construction Specifications, subsection 703-04

General Equipment

a. 3/16 inch diameter pop rivets
b. friction or plastic tape
c. 6 inch diameter drive pipe with necessary couplings
d. steel cap for 6 inch diameter drive pipe
e. cement to fasten couplings to plastic grout pipe

1.3 Construction Details

1.3.01 Drilling

The Contractor shall install and clean out the 6 inch diameter drive pipe to a minimum depth of 5 feet below the installation grade (bottom of cap beam, see Fig. 1 - Typical Slope Indicator Installation).

This 6 inch diameter drive pipe shall extend to the final pavement grade. The Contractor shall have the option of installing this pipe in one or two lengths. The 6 inch diameter drive pipe will remain in place as part of the permanent installation.

The hole into which the slope indicator casing is installed shall be drilled under another contract item.

1.3.02 Installation of Slope Indicator Casing

A. Grouting Check Valve. The grouting check valve (as detailed in Fig. 2 - Typical Grouting Check Valve Detail or equivalent) shall be installed in the first 10-foot section of slope indicator casing. Four 3/16 inch diameter holes shall be drilled through the slope indicator casing and the mating part of the check valve assembly. The check valve assembly shall be pop riveted in place and the rivet holes sealed using plastic tape. The rivets shall not be installed through the grooves of the slope indicator casing.

B. Assembly of Slope Indicator Casing

1. The 10 foot section of S.I. casing with the check valve assembly attached shall be lowered into the hole and secured in place with a clamp so that the next section of casing can be attached.

2. A plastic coupling shall be attached to the bottom of each subsequent 10 foot section of S.I. casing in the manner detailed for the installation of the grouting check valve assembly.

3. The aligning tool, with a length of rope attached shall be inserted (half the tool's length) into the top of the section of S.I. casing in the hole. The rope shall be threaded through the section of casing to be added. This section of casing shall then be lowered over the protruding aligning tool until it butts with the first section of casing in the hole. The two sections shall then be riveted as previously detailed. The casing shall then be lowered 10 feet and the aligning tool shall be withdrawn to the top of the casing so as to protrude 1/2 its length above the casing. This procedure shall be repeated until the slope indicator casing reaches the bottom of the hole.

4. The top of the S.I. casing at this point shall be at or above the proposed final pavement grade.

5. The S.I. casing shall then be oriented so that one set of opposite grooves is parallel to the anticipated direction of soil foundation movement as directed by the Engineer.

6. The S.I. casing shall be cut off so that the top will be approximately 2 inches below final pavement grade (see Fig. 1 - Typical Slope Indicator Installation).

C. Grouting Slope Indicator Casing in Place

1. The grout shall consist of 1 part Portland cement and 1 part water or as otherwise approved by the Engineer. Grout shall be transmitted to the bottom of the hole via the 1 inch diameter rigid plastic pipe.

2. A threaded coupling shall be cemented to the bottom of the section of grout pipe which shall then be lowered into the S.I. casing (see Fig. 2 - Typical Grouting Check Valve Detail).

3. Subsequent sections of grout pipe shall be added in convenient lengths joined together by cemented slip couplings until the grout pipe reaches the bottom of the S.I. casing.

4. The grout pipe shall be screwed into the check valve that was installed on the lowest section of S.I. casing.

5. The grout pipe shall be cut to a convenient height and a quick disconnect valve shall be cemented to it.

6. Prior to grouting operations, the Contractor shall employ measures necessary to insure that the S.I. casing will not "float" out of the hole.

7. The S.I. casing shall be grouted in one stage from the bottom to the top. The grout shall be pumped through the grout pipe until it surfaces at the top of the hole around the outside of the S.I. casing.

8. The steel boring casing may be removed after completing the grouting operation or it may be removed in increments as the grout is pumped. However, the grout shall be kept within 5 feet of the surface as the boring casing is removed. At the completion of the S.I. casing installations, the [redacted] boring casing shall be completely removed.

9. After the grout is installed and before it sets, the grout pipe shall be removed from the check valve assembly and raised up approximately 1 foot. Clean water shall be pumped into the grout pipe to clean it of grout and the slope indicator casing shall be cleaned of all grout by surging. The grout pipe shall then be removed from the slope indicator casing.

10. If the grout cannot be maintained within 5 feet of the surface as the boring casing is being removed or at the surface when the boring casing is completely removed then the S.I. casing shall be removed from the hole. The hole shall be re-drilled to the original lower limit of the boring and the S.I. casing re-installed.

Note

The Contractor's attention is directed to fact that voids exist in the materials behind the existing walls. It is the Contractor's responsibility to insure that the slope indicator casing is completely and securely attached to the insitu soil/rock/boulder mass. Special methods and/or materials may be necessary to complete this work. If a cement mortar or grout material is used to seal the sides of the hole, the mix shall be such that the one day strength does not exceed 100 psi.

Prior to the start of work under this item, alternate methods and procedures proposed by the Contractor for properly installing the S.I. casing will be considered and if approved by the Engineer will be allowed to be used.

D. Capping and Enclosing Slope Indicator Casing

1. The protective cap shall be installed at the top of the S.I. casing.

2. The 6 inch diameter drive pipe shall be capped. The cap shall be flush with the pavement surface after paving is completed.

1.4 Method of Measurement

The quantities to be paid for shall be the actual number of linear feet of slope indicator casing satisfactorily installed in accordance with this specification measured from the final pavement grade to the bottom of the slope indicator casing.

1.5 Basis of Payment

The unit price bid per linear foot of slope indicator casing shall include the cost of furnishing and installing the 6 inch diameter drive pipe and slope indicator casing, grouting (including special methods and/or materials as necessary), capping the slope indicator casing and 6 inch diameter drive pipe, maintaining the slope indicator casing throughout the life of the contract and the cost of furnishing all labor, materials and equipment necessary to complete the work as prescribed in this specification.

No additional payment will be made for labor, materials and/or equipment required to seal a hole where grout is lost and re-drilling is necessary.

Item No.		Item	Unit
17203.98	-	Slope Indicator Casing	Lin. Ft.

INSTALLATION OF SLOPE INDICATOR CASINGS

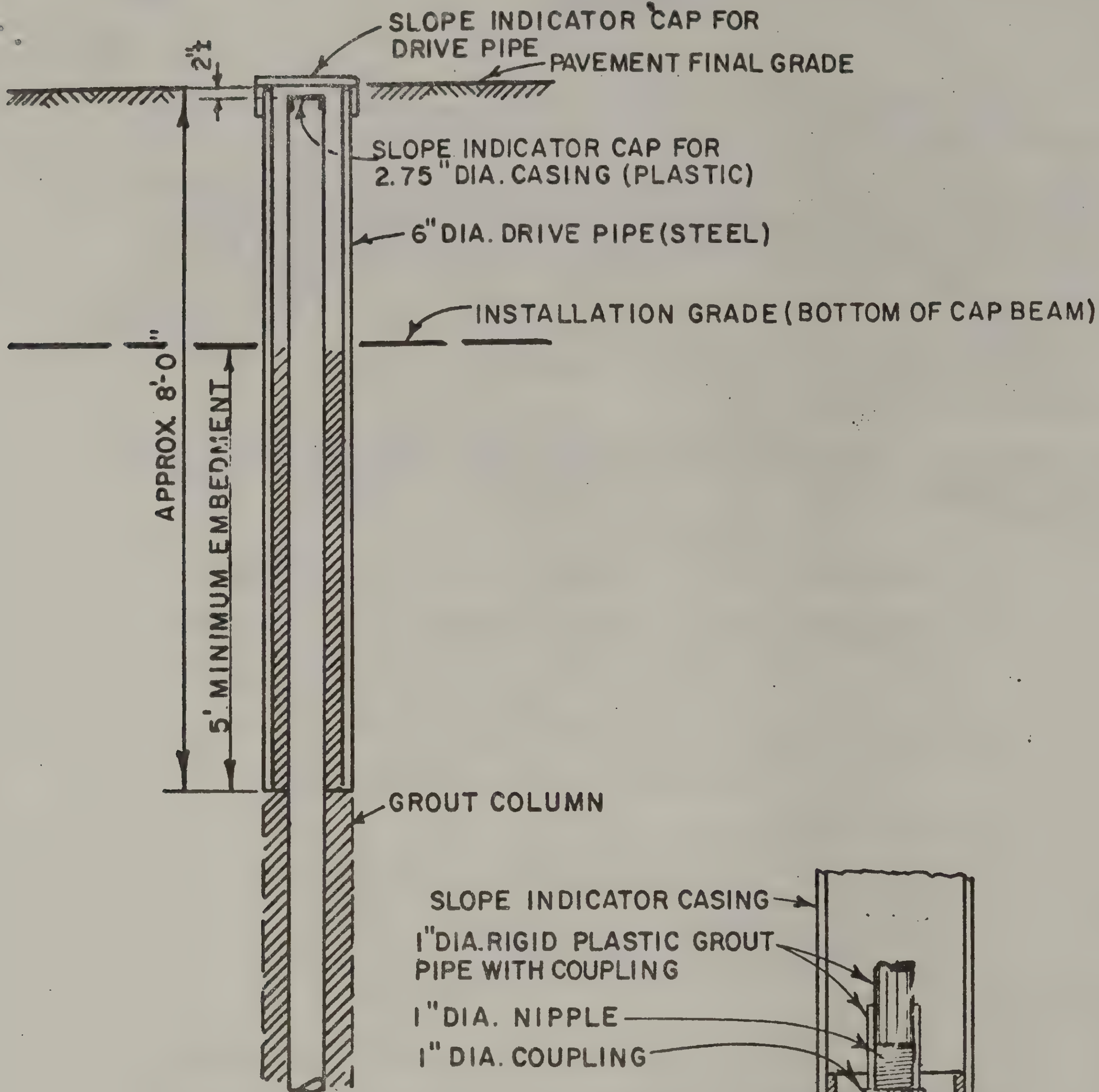


FIGURE 1

YPICAL SLOPE INDICATOR INSTALLATION

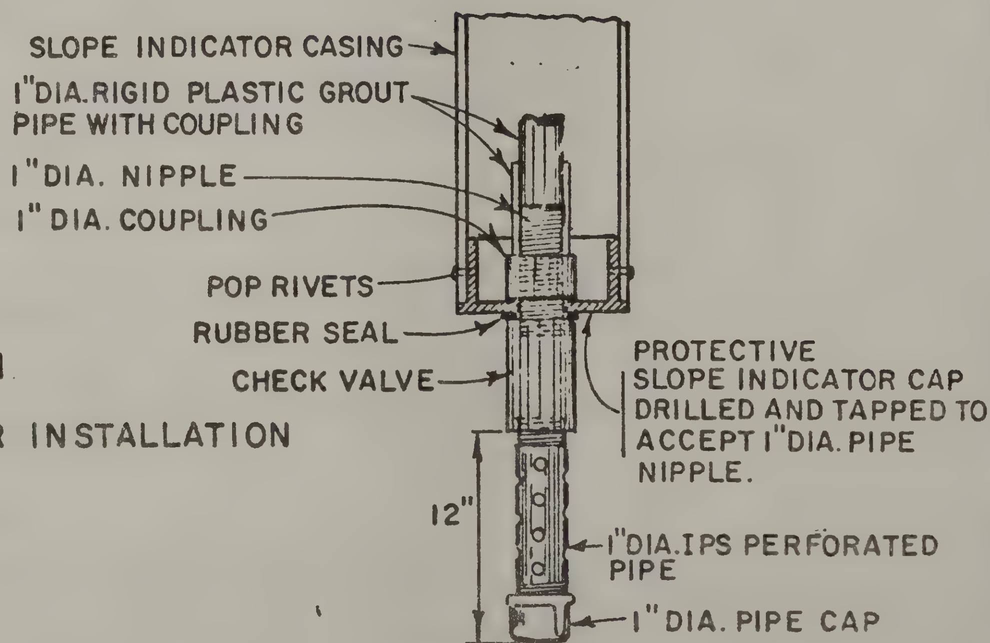


FIGURE 2

TYPICAL GROUTING CHECK VALVE DETAIL

ITEM 17203.9801 SUPPLYING NEW READOUT EQUIPMENT FOR SLOPE INDICATORS AND TILTMETERS

2.1 Description

Under this specification, the Contractor shall supply and maintain new slope indicator and tiltmeter systems. After the completion of the contract, all equipment specified under this item shall become the property of the State. The equipment shall be in proper working order when turned over to the State.

2.2 General

The slope indicator and tiltmeter system shall consist of the following component parts. The model numbers are component parts provided by:

Slope Indicator Company (Sinco)
3668 Albion Place North
Seattle, Washington 98103

and are included for reference purposes only.

	Sinco Model No.	Qty.
Digitilt Indicator w/manual component switch	50306	1
Digitilt Biaxial sensor	50325	1
Digitilt tiltmeter sensor	50322	1
Ceramic tilt plates	50323	25
Control cable	50610	100 ft.
Pulley assembly w/cable hole	51117	1

2.3 Materials Requirements

All materials must conform to the specifications of or be the equivalent to the equipment listed above as provided by the Slope Indicator Company. All slope indicator-related components must be compatible with each other for each installation. Any equipment desired to be used other than that listed above must receive prior approval by the Director of the Soil Mechanics Bureau.

2.4 Basis of Acceptance

This material will be accepted on the basis of the manufacturer's specifications for the system subject to the approval of the Director of the Soil Mechanics Bureau.

2.5 Basis of Payment

The bid price shall include all costs of furnishing and maintaining the specified equipment.

Item No.	Item	Unit
17203.9801 -	Supplying New Readout Equipment for Slope Indicators and Tiltmeters	Lump Sum

17605.2599 HORIZONTAL DRAINS

DESCRIPTION

This work shall consist of the installation of horizontal drain pipe at the locations shown on the plans or as specified by the Engineer. This item includes the work for advancing the boring, however, the boring equipment shall be furnished under the item Furnishing Equipment and Site Preparation for Installing Horizontal Drains.

MATERIALS

The PVC material for the drain pipe shall meet the requirements of ASTM Designation: D1784 for "Rigid Poly (Vinyl Chloride) Compounds and Chlorinated Poly (Vinyl Chloride) Compounds," Class 12454-B.

The molded or extruded pipe shall conform to ASTM Designation: D1785 for "Poly (Vinyl Chloride) (PVC) Plastic Pipe," Schedule 80, PVC 1120.

The solvent cement shall meet the requirements of ASTM Designation: D2564 for "Solvent Cements for Poly (Vinyl Chloride) (PVC) Plastic Pipe and Fillings."

The dimensions and tolerances of the pipe shall conform to ASTM Designation: D1785.

Nominal Pipe Size (Inches)	Outside Diameter (Inches)	Wall Thickness Schedule 80 (Inches)
1 1/2	1.900 $\pm$ .006	0.200 + 0.024

The pipe shall be flush coupled, and shall be continuously slotted except at the outlet of each drain. The slots shall be in a tri-slot configuration, i.e., three individual slots around the perimeter of the pipe. The slot width and number of slots per foot of pipe shall be as specified in the proposal.

Basis of Acceptance. All pipe, fittings and solvent cement shall be accepted on the basis of the Manufacturer's certification that the material conforms to this specifications.

CONSTRUCTION DETAILS

The drain pipe shall be placed in a cased boring that is inclined between 0 and 20 degrees above the horizontal. The actual inclination of each cased boring shall be specified in the field by the Engineer at the time of installation. The inserted end of the drain pipe shall be plugged in a manner acceptable to the Engineer. The drain pipe shall be slotted except for the last 20 feet of the outlet end. After completing the required drilling, the length of drain pipe shall be cemented together and inserted into the casing such that approximately two feet of the drain pipe will be exposed after the drill casing has been removed.

METHOD OF MEASUREMENT

Horizontal drains shall be measured for payment by the actual number of lineal feet of plastic drain pipe satisfactorily installed and approved by the Engineer.

20-

BASIS OF PAYMENT

The unit price bid per lineal foot shall include the cost of all labor and materials necessary to furnish and install the drain pipe. All tools, and equipment necessary to complete the work shall be paid separately under the Item for Furnishing Equipment and Site Preparation for Installing Horizontal Drains.

17605.2699 FURNISHING EQUIPMENT AND SITE PREPARATION FOR INSTALLING HORIZONTAL DRAINS

DESCRIPTION

This work shall consist of furnishing all equipment necessary for progressing casings for horizontal drain pipe as well as labor, material and equipment required to construct, remove and regrade any necessary access roads, working platforms and drainage blankets. The work for progressing the casings shall be included in the item for Horizontal Drains.

MATERIALS

Equipment Requirements

The Contractor shall furnish rotary drills and auxiliary equipment, including water pumps, to progress a cased boring through which the horizontal drain pipe will be inserted.

a) The Rotary drill for installing the casing shall be powered by an internal combustion engine that is rated at a minimum of 90 horsepower at the flywheel.

b) The drill head shall be mounted so that the travel direction of the drill casing is parallel in a plan view to the direction of travel of the crawler tractor.

c) The minimum inside diameter of the drill casing shall be sufficient to allow the plastic drain pipe to be inserted.

Working Platform and Drainage Blanket

The material furnished for this work shall be visually inspected by the Engineer and approved if satisfactory for the intended use. The State reserves the right to reject material not meeting the following requirement: a crushed stone or crushed gravel blend by weight consisting of 50% No. 1 and 50% No. 2 size as described in Table 703-5.

CONSTRUCTION DETAILS

The Contractor shall construct all necessary work platforms and access roads. At the completion of the installation of the drain pipes, the working platforms shall be regraded to the limits shown in the proposal and the access roads shall be regraded as nearly as possible to the original contours.

METHOD OF MEASUREMENT

Payment will be made at the lump sum bid for this item.

BASIS OF PAYMENT

Progress payments will be made in the following manner:

Fifty percent (50%) of the amount bid will be paid when the access roads and work platforms are constructed and the equipment for installing casings is operating satisfactorily at the site.

Twenty-five percent (25%) will be paid when all of the casings are properly installed and removed.

The remaining twenty-five percent (25%) will be paid when the site is graded to its original or designated condition and all equipment and materials are removed.

SUGGESTED SPECIAL SPECIFICATION FOR
CO. RD. 3 (FLY ROAD) AND PCRR GRADE SEPARATION
PSC 25928 PIN 2750.21-101

ITEM 02203.1199 HOSE SETTLEMENT GAGE

Description

The Contractor shall furnish, install and maintain in good operating condition the Hose Settlement Gage and all appurtenances required by this specification and the contract plans at the location shown on the plans, or at locations ordered by the Engineer.

Materials

Materials shall be provided as shown on the plans.

The PVC pipe, couplings and solvent cement shall meet the material requirements of Item 706-15 PVC Plastic Drain Pipe System unless modified on the plans.

The bedding sand shall be any fine aggregate meeting the gradation specifications of Item 703-07 Concrete Sand.

The concrete used to construct the terminal blocks shall meet the material requirements of Item 601.02 Class B Concrete for Structures.

Construction Details

The Hose Settlement Gage shall be installed according to details shown on the plans and the following construction procedures.

Prior to embankment placement, but after topsoil removal operations if required, a shallow trench with a smooth bottom shall be excavated at the locations indicated on the plans. A three inch thick bedding layer of sand shall be placed in the trench. This layer shall be shaped to conform to the plan grade.

Connect the PVC pipe using slip couplings and solvent cement and lay it in the trench threading the 1/8 inch diameter flexible non-corrosive wire rope (airplane cable or equivalent) through the pipe as the work proceeds. This cable shall be one continuous unspliced length. An 'eye' is to be formed at each end of the cable using copper or aluminum sleeves crimped with a hand swagger.

Cut and thread the PVC pipe at the terminal blocks for the placement of the end caps.

The installation of the PVC pipe, cable and terminal blocks shall be inspected and approved by the Deputy Chief Engineer (Technical Services) based on an evaluation by a Departmental Soils Engineer.

The PVC pipe shall not be covered until the Engineer has surveyed and recorded the elevation of the PVC pipe and the position of all couplings and measured the total length of the PVC pipe.

Backfill the trench as shown on the plans.

The readout device and personnel required to take readings will be supplied by the State

Method of Measurement

The quantity to be paid for under this Item shall be the number of linear feet of hose settlement gage installed in accordance with the plans or as directed by the Engineer.

Basis of Payment

The unit price bid per foot of hose settlement gage installed shall include the cost of furnishing all labor, material, equipment maintenance and incidentials necessary to complete the work including all excavation and backfill of the trench and construction of and backfill of the terminal blocks. No payment will be made under any other item for the work necessary to complete this Item. Payment will be made upon satisfactory completion of the installation. The hose settlement gage and appurtenances installed by the Contractor shall remain in place and become the property of the State at the completion of the contract.

Damage to the hose settlement gage subsequent to installation, during the life of the contract, shall be repaired by the Contractor to the satisfaction of the Engineer.

Item 17648.26 - Vane Shear Test in Soil

Description.

This work shall consist of making vane shear tests in soil.

Materials.

The Contractor shall provide two sets of new vane shear equipment to be used by the Engineer on this contract. The vane shear sets shall include all the necessary vanes, drill rods, bearing guides, torque assembly, force gages, and all other equipment necessary to complete the vane shear test. All of the equipment except for the vanes and vane rods shall be equal to the vane shear apparatus manufactured by Acker Drill Company. The vane and vane rod equipment shall conform to the details shown on attached drawings numbered SM1604RI (Vane #3 - 3" x 6") and SM1611RI.

Construction Details.

These tests shall be conducted at selected depths in the soil strata as directed by the Engineer. Each test shall consist of a series of two vane shear strength determinations taken at depth intervals of twelve (12) inches. Each series of two vane shear strength determinations shall constitute one vane shear test for payment purposes. Most of the vane shear strength determinations will be obtained by utilizing a shearing strain rate of 1.67% per minute. Up to 10 special determinations will be obtained using lower strain rates. It is estimated that up to 1½ hours of additional time will be required per test for this purpose. Three "Blank Tests" per hole shall also be performed to determine the frictional resistance characteristics of the test apparatus when and as directed by the Engineer. Blank tests are performed in the same manner as vane shear tests except that the vane is omitted. Special tests and blank tests when requested shall be considered a part of the entire vane shear test procedure. The Contractor shall assemble the apparatus including bearing guides at depth intervals specified by the Engineer and placed at the correct elevation in the drilling hole. It will be necessary to assist the Engineer in conducting the test by recording test readings and furnishing other assistance necessary. The Engineer or his designated representative, however, will operate the Vane Shear Test Equipment during the shearing strength phase.

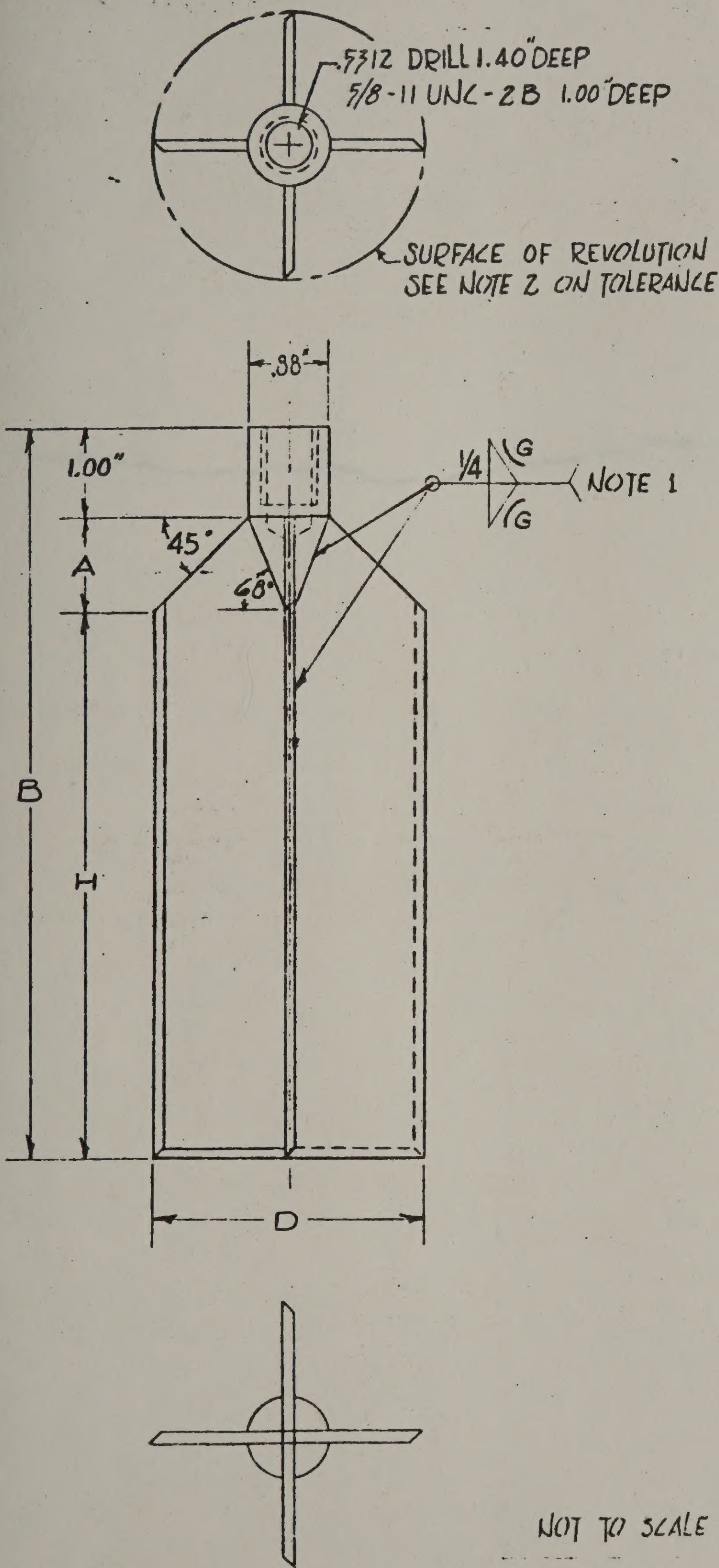
After a series of two determinations have been completed, the Contractor shall obtain soil samples of the materials encountered at the elevation of the vane shear tests.

Method of Measurement.

Payment will be made for the actual number of vane shear tests conducted, as directed by the Engineer.

Basis of Payment.

The unit price bid for this Item shall cover all labor, materials, supplies, plant, tools, including the vane shear apparatus, and incidentals required for assisting in the vane shear tests, special tests, blank tests and obtaining soil samples.



MATERIAL SPECIFICATIONS
VANES - TYPE 301 STAINLESS STEEL SHEET, 13 GA.
HUB - TYPE 316 STAINLESS STEEL COLD DRAWN ROD.

DIMENSION	VANE 1	VANE 2	VANE 3	VANE 4
D	2"	2½"	3"	2"
H	4"	5"	6"	8"
A	.56"	.81"	1.06"	.56"
B	5.6"	6.8"	8.1"	9.6"

NOTES

1. FILLET WELDS SHALL BE MACHINE GROUND TO A UNIFORM CONCAVE RADIUS OF .25" BY MEANS OF AN APPROPRIATE WHEEL. FINISHED WELDS ARE TO BE FREE OF ALL DEFECTS.

2. FINAL FINISHED UNITS SHALL DESCRIBE A CONCENTRIC CYLINDRICAL SURFACE OF REVOLUTION WITH A DIAMETRAL TOLERANCE OF .05" TOTAL WHEN ROTATED IN THE AXIS OF THE HUB THREAD CONNECTION.

REVISIONS			
1	REMOVED BEVEL AREA "A"	1/16/62	J.H.
2			
3			

NOT TO SCALE

FIELD VANE SHEAR APPARATUS

VANE DETAILS

9/26/61

D.J.B.

Wm. P. Hofmann

SM 1604 R1